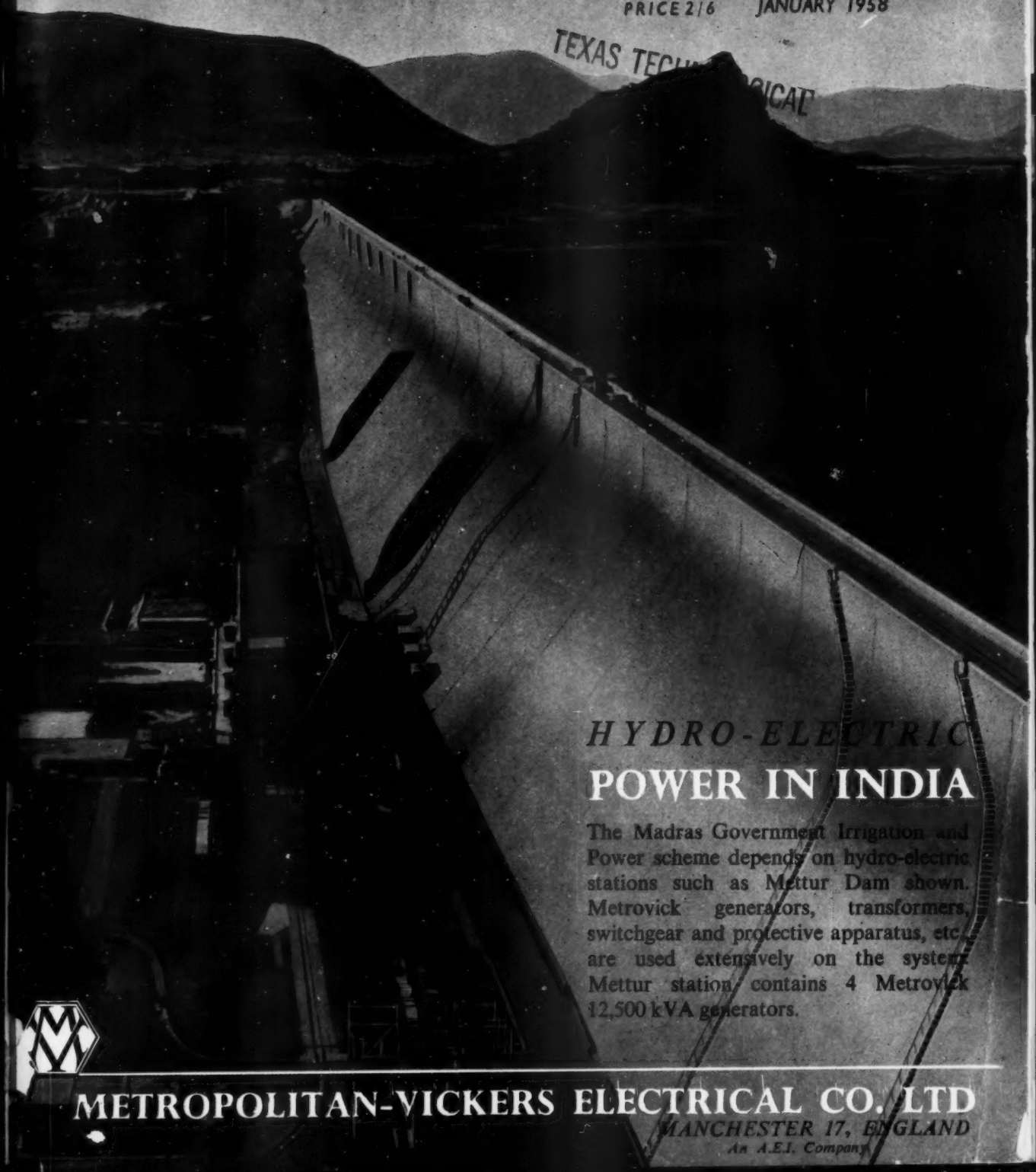
Water Power

INTERNATIONAL DEVELOPMENT OF HYDRO-ELECTRIC POWER

PRICE 2/6 JANUARY 1958

TEXAS TECH[illegible]ICAL



HYDRO-ELECTRIC

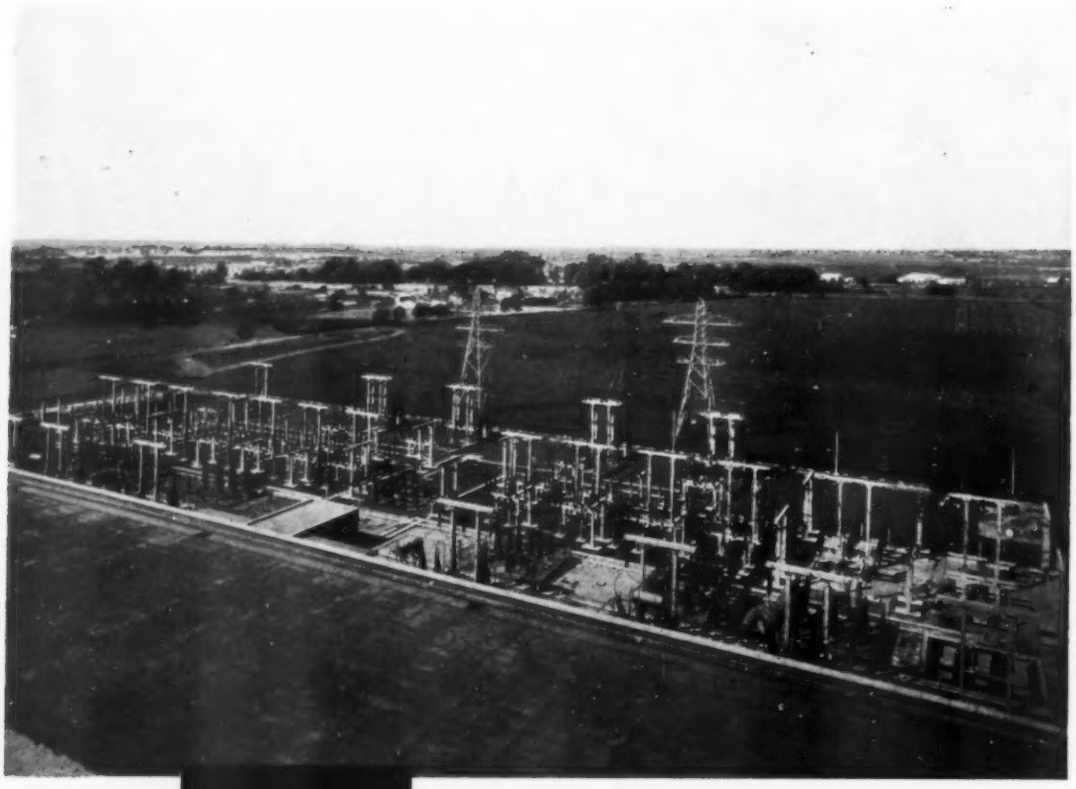
POWER IN INDIA

The Madras Government Irrigation and Power scheme depends on hydro-electric stations such as Mettur Dam shown. Metrovick generators, transformers, switchgear and protective apparatus, etc., are used extensively on the system. Mettur station contains 4 Metrovick 12,500 kVA generators.

METROPOLITAN-VICKERS ELECTRICAL CO. LTD

MANCHESTER 17, ENGLAND

An A.E.I. Company



GOLDINGTON POWER STATION

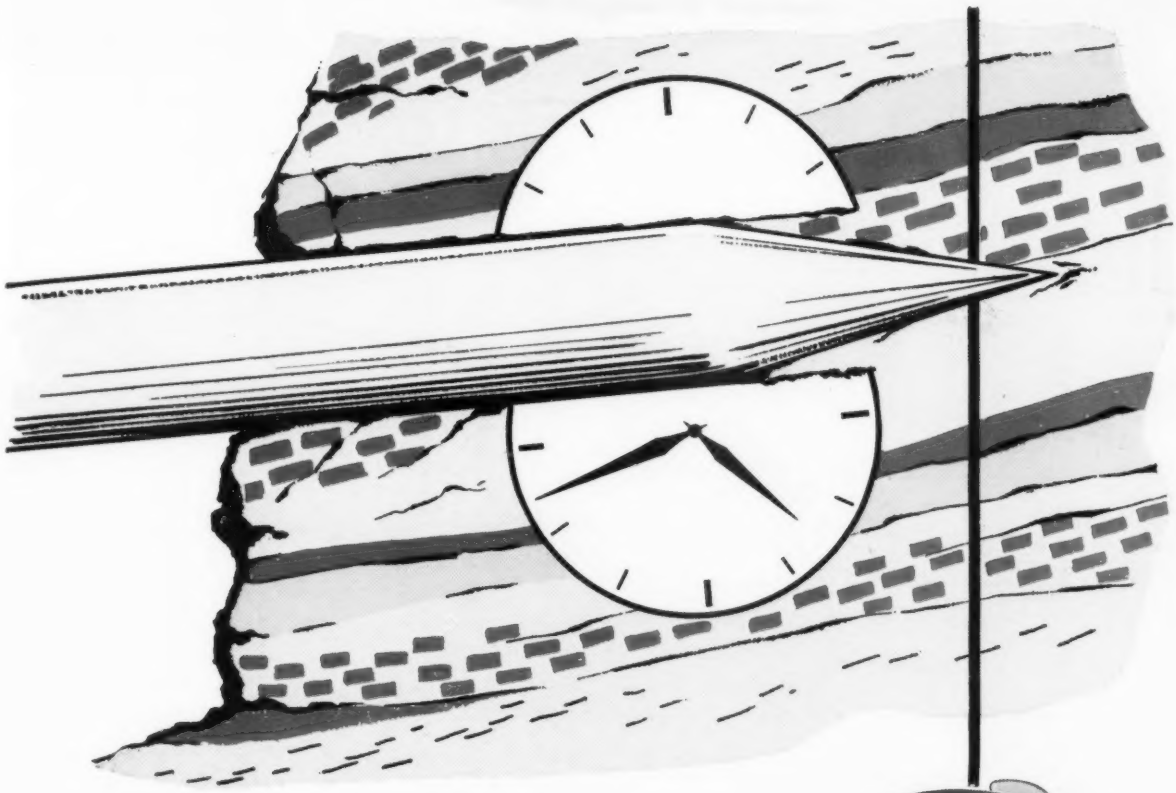
Consulting Engineers : Messrs. Merz and McLellan

Photograph by courtesy of the CEA Eastern Division

The main 132-kV 2500-MVA air-blast switchgear with its associated control-equipment as well as the 3·3-kV oil-break switchgear and 415-volt air-break switch and fuse gear for the station auxiliaries at Goldington Power Station was supplied by Reyrolle.

Reyrolle

A. REYROLLE & COMPANY LIMITED - HEBBURN - COUNTY DURHAM - ENGLAND



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Hard, slogging blows at a rate of 1350 a minute. A 5½-inch stroke. Amazingly little recoil. A shape that makes for slick, easy handling even in tight corners. All this you get with Holman's tough PW pick. And more. For this pick, one of a range of wet and dry, has earned a wide reputation for taking heavy punishment without a murmur. Job after job after job.

It saves money in *two* ways: by getting on with the work at speed and by not letting you in for high running costs. It's the breakdowns you *don't* have that make it worth more than its price: a feature common to the whole range of Holman wet and dry picks.

Proof of the pudding: try out a Holman pneumatic pick on your toughest, awkwardest job. Your Holman agent will send you one right now.

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pays . . . with its life

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The Holman Organisation is world-wide. It includes the Climax Rock Drill & Engineering Works Ltd., and has agents, branches and technical representatives throughout the United Kingdom and in 84 other countries.

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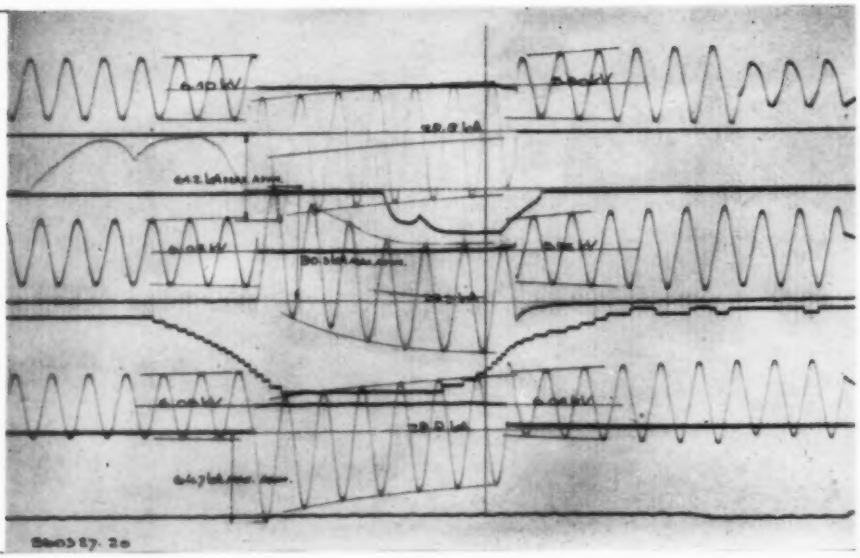
HJ27

mt **type** **New series**

# of medium voltage circuit breakers



2017 - 56-03

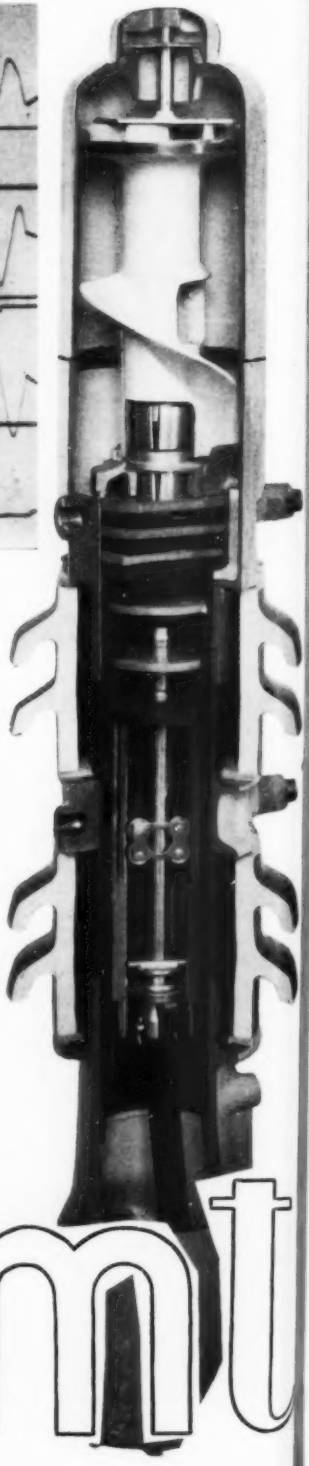
TYPE TESTS: QUALITY WARRANTY

The ratings of a circuit breaker can be validly supported only by suitable **type tests** proving the circuit breaker behaviour under the worst conditions which must be faced on service.

The MT circuit breakers of SCARPA & MAGNANO underwent, for each service voltage, the type tests required by the **I.E.C. International Standards**, at the official Laboratory of KEMA, in Holland.

The certificates proving the favourable result of these tests are by SCARPA & MAGNANO kept at disposal of customers.

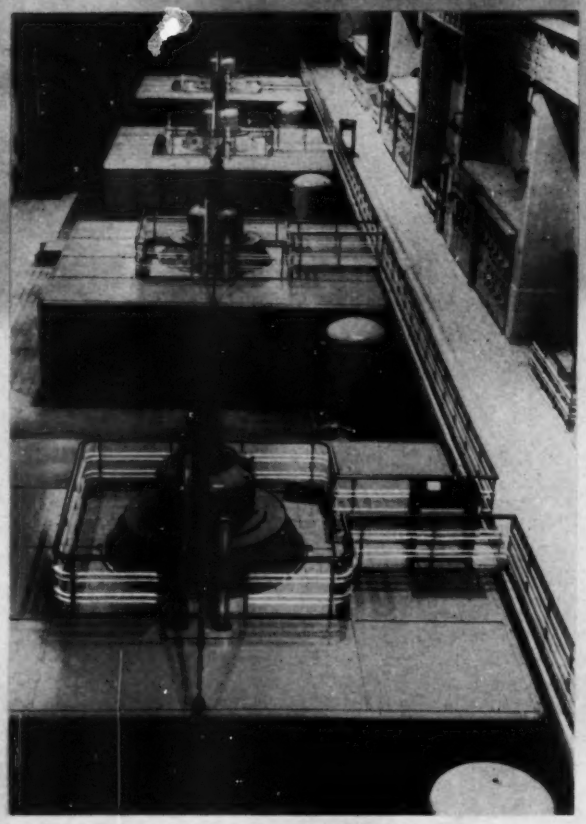
CERTIFICATE



mt

Scarpa & Magnano

Savona - Italy



Owen Falls

This is one of the first large hydro-electric schemes developed to meet the rapidly growing power demand in Africa.

Boving & Co. are supplying the ten Kaplan turbines to be installed to give a total capacity of 150,000 kW. Four turbines driving B.T.H. generators are already in commercial service, furnishing power to many new industries in Uganda made possible solely through power available from Owen Falls.

Boving

A COMPANY LIMITED

WATER POWER ENGINEERS

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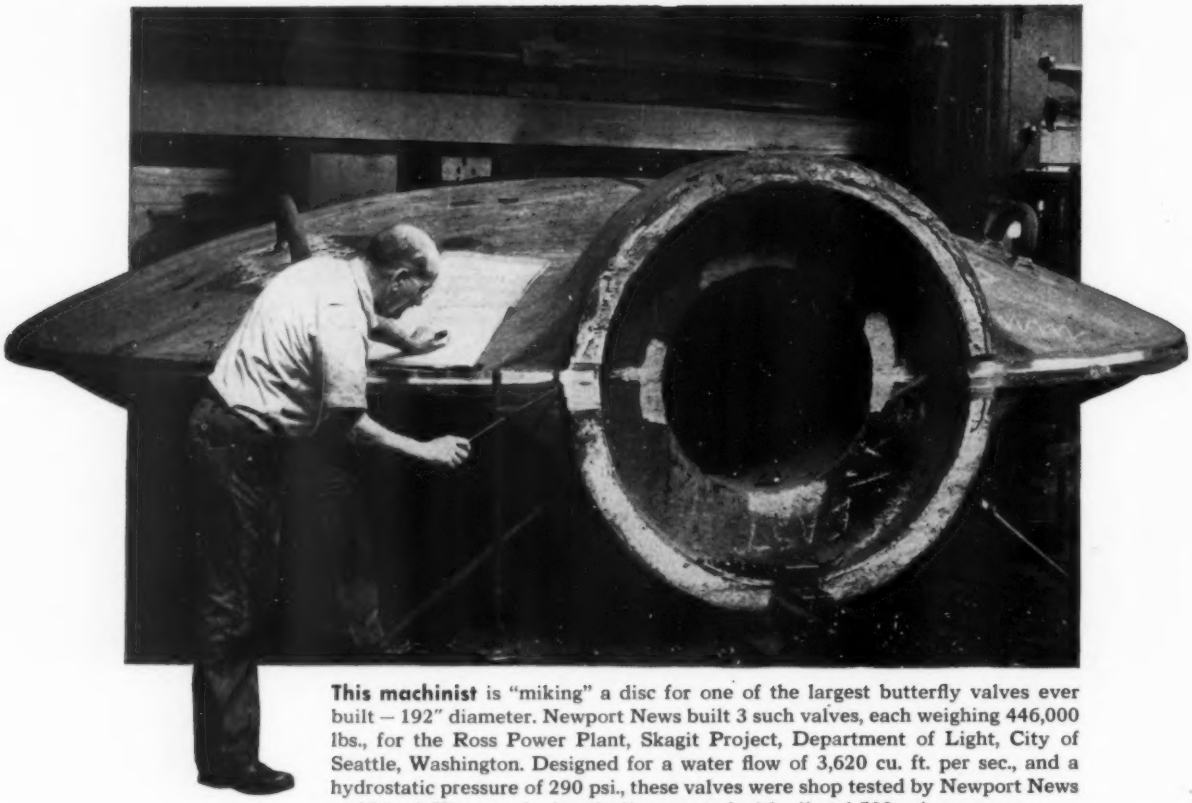


This machinist is "miking" a disc for one of the largest butterfly valves ever built – 192" diameter. Newport News built 3 such valves, each weighing 446,000 lbs., for the Ross Power Plant, Skagit Project, Department of Light, City of Seattle, Washington. Designed for a water flow of 3,620 cu. ft. per sec., and a hydrostatic pressure of 290 psi., these valves were shop tested by Newport News at 450 psi. They are hydraulically operated with oil at 1,500 psi. pressure.

Birth of a **200-ton** Butterfly

This disc for a 16-foot butterfly valve reflects two basic advantages of Newport News fabrication...

First, it exemplifies the *careful attention* Newport News craftsmen give to every detail. And secondly, it attests to the *quality* with which Newport News produces in massive equipment for public utilities and allied industries... due to Newport News' high integration of skill and production facilities.

Additional advantages accrue to customers from extensive work conducted in Newport News' testing laboratories on problems related to water power equipment.

Avail yourself of the engineering talent, along with the specialized production techniques and the skill of Newport News craftsmen operating vast steel fabricating shops, five huge machine shops, drop forging and die facilities along with acres of brass, iron and steel foundries.

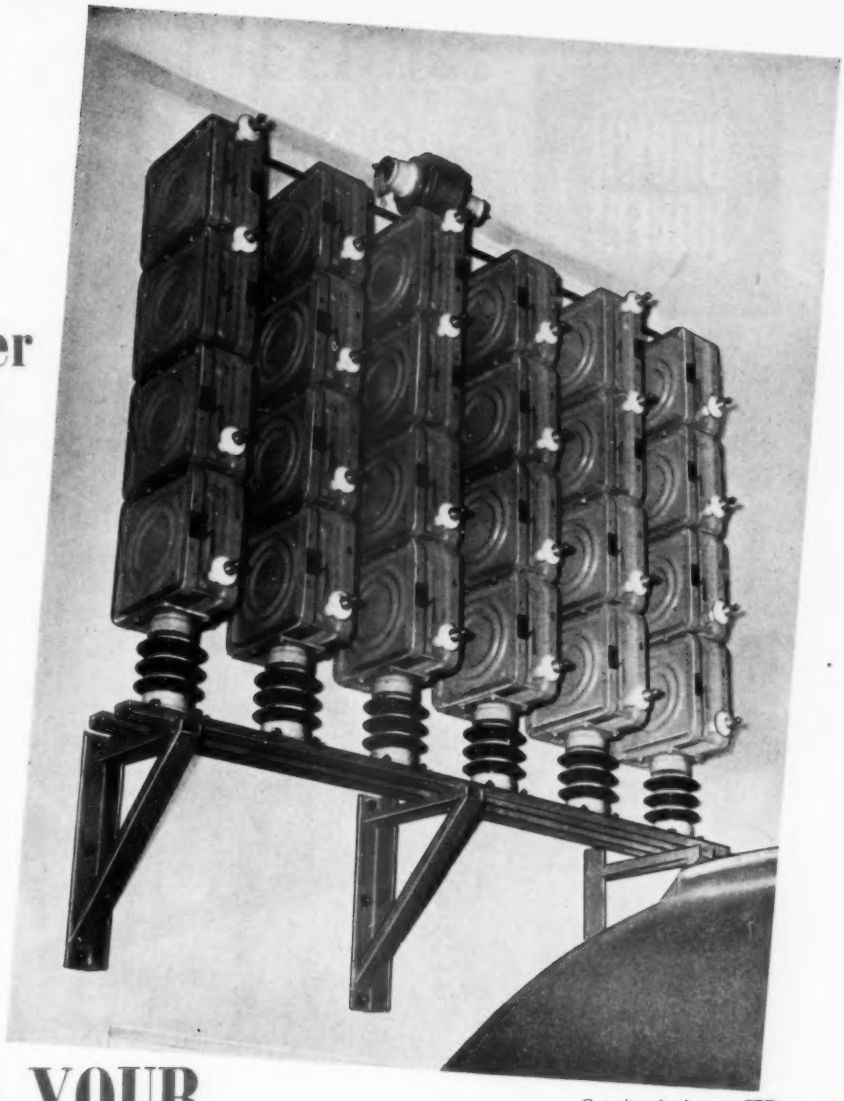
Let us bid on equipment for your present or future projects. If you are not familiar with the way Newport News can help you, write for our illustrated booklet entitled "Water Power Equipment" ...it's yours for the asking.

NEWPORT NEWS

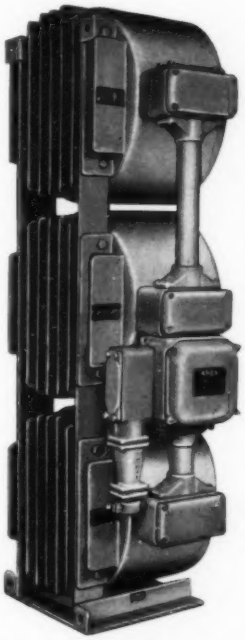
Shipbuilding and Dry Dock Company

Newport News, Virginia

Are YOU penalized for reactive power consumption?



Capacitor bank type CTD, 480 kVAr, 6·3 kV, 50 c/s, installed in a Norwegian pulp mill.



Capacitor type CTA, 87 kVAr, 400 V, 50 c/s, mounted in frame for fixing to wall or floor. Note the ample terminal enclosure.

Is YOUR electrical equipment being overloaded?

There is an unabating increase in the demand for electric power and many power supply systems are being worked to the point of failure.

ASEA Capacitors furnish an expedient and inexpensive means of improving your power situation.

The capacitors are designed as standard units which can be built up into banks of requisite size. Banks may be enlarged by adding units, or the rating altered by transferring units from one bank to another, according to prevailing supply conditions.

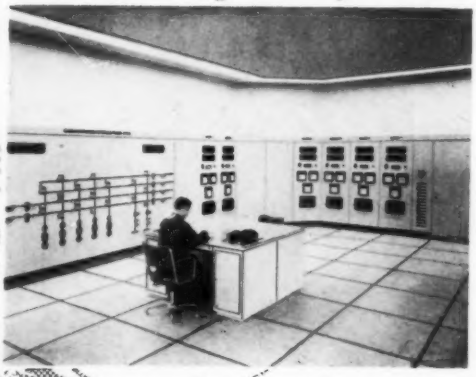
Consult ASEA for expert advice on your power problems. Capacitors in all sizes for 230, 400, 440 and 500—550 V can be supplied from stock.

ASEA Capacitors

- ★ **reduce all transmission losses**
- ★ **improve voltage conditions in the supply mains**
- ★ **permit higher active loading of generators, cables and transformers**
- ★ **enable the power producers to supply more kilowatts**
- ★ **entail lower costs for energy consumed**
- ★ **obviate penalties for excessive consumption of reactive power and energy**
- ★ **pay for themselves in a very short time merely by the reduction in penalties for reactive power**

Västerås ASEA **Sweden**

BROWN BOVERI



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are an ideal means of communication for electricity undertakings.

All over the world they are in use with telephone, teleprinter, telemetering and regulation systems; also transmitting signals for the remote control of sub-stations and generating plant, and for coupling high-speed distance protection relays.

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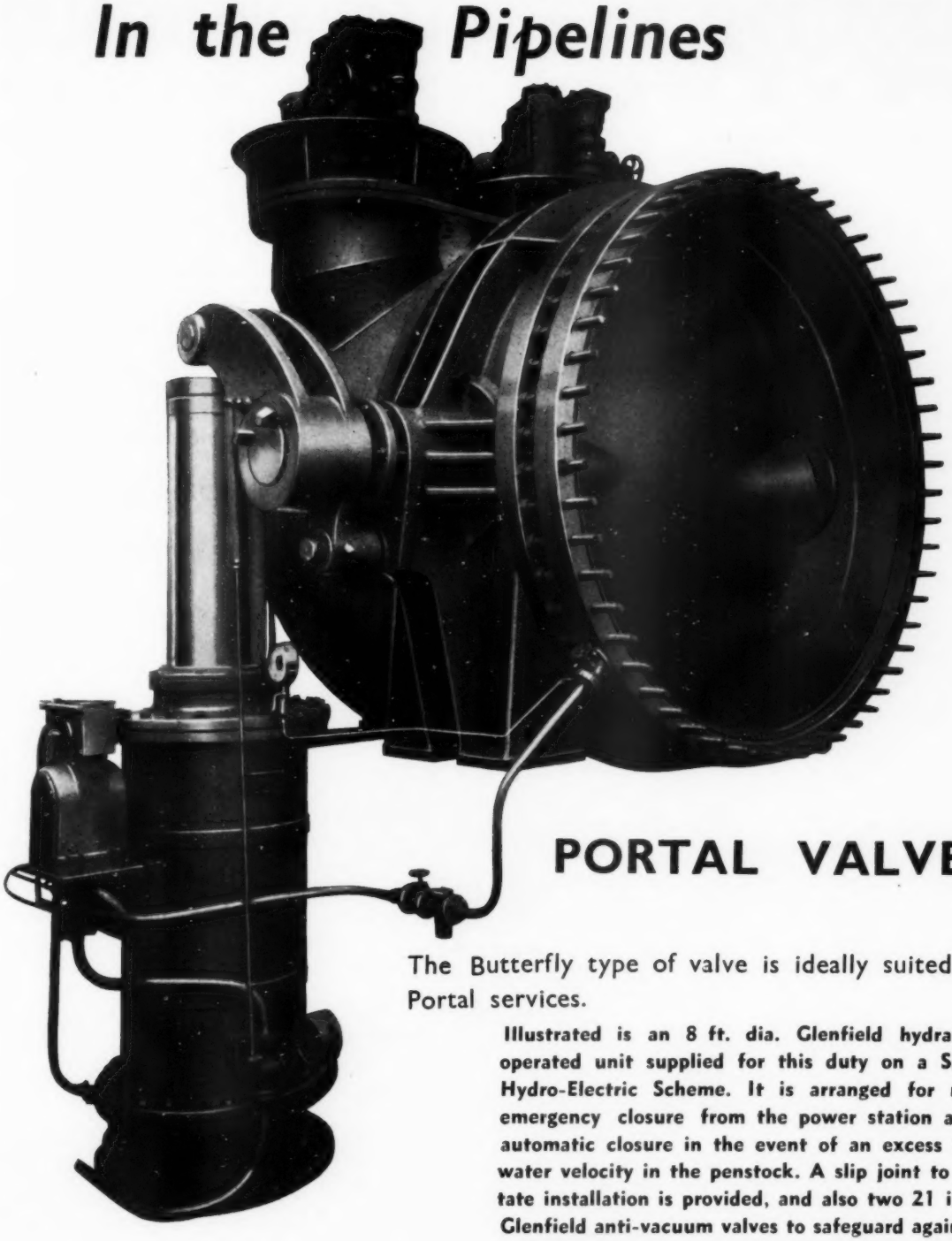
In the Pipelines



PORTAL VALVES

The Butterfly type of valve is ideally suited for Portal services.

Illustrated is an 8 ft. dia. Glenfield hydraulically operated unit supplied for this duty on a Scottish Hydro-Electric Scheme. It is arranged for remote emergency closure from the power station and for automatic closure in the event of an excess rise of water velocity in the penstock. A slip joint to facilitate installation is provided, and also two 21 in. dia. Glenfield anti-vacuum valves to safeguard against the formation of vacuum in the penstock on the downstream side, when the main valve is closed in emergency.

Glenfield

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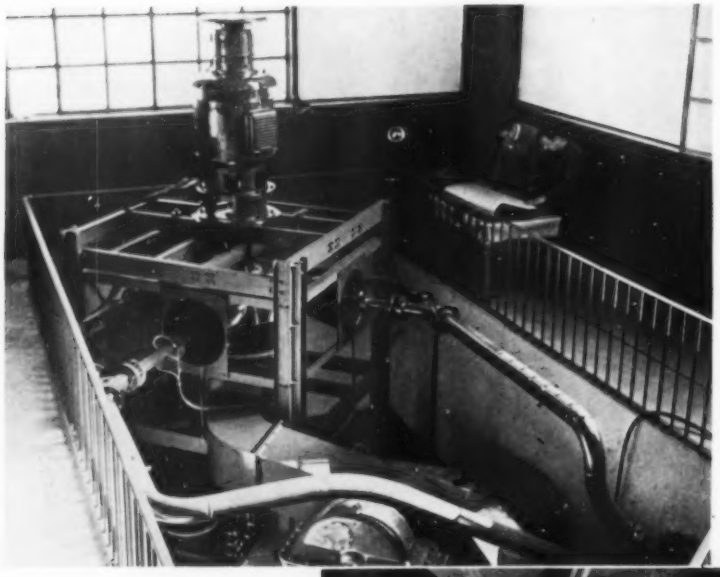
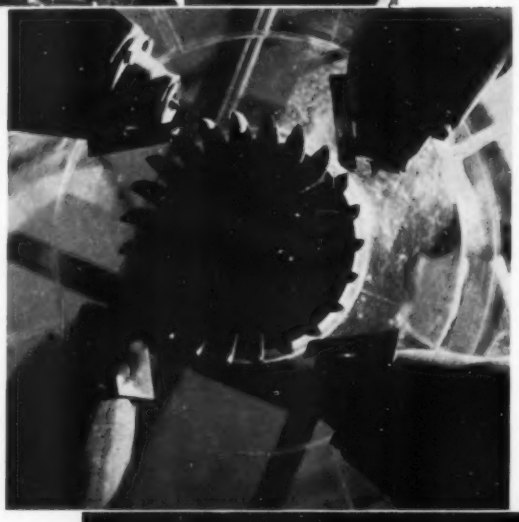
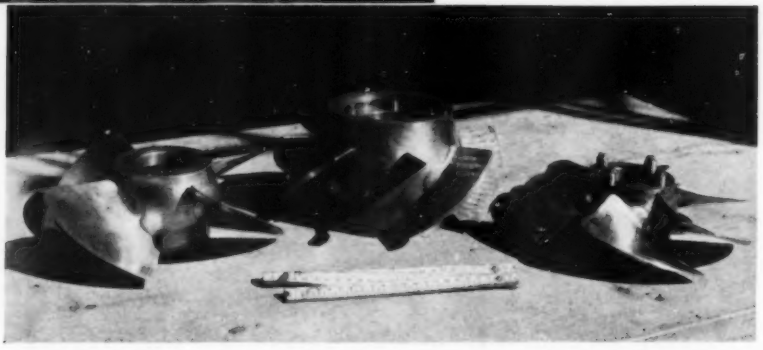
KAPLAN ★ FRANCIS ★ PELTON ★ GOVERNORS ★ VALVES



Franco Tosi testing facilities on scale model turbines installed in A.F.L. Falck Hydroelectric Station of Teglia (Tuscany district Apennine, Northern Italy).

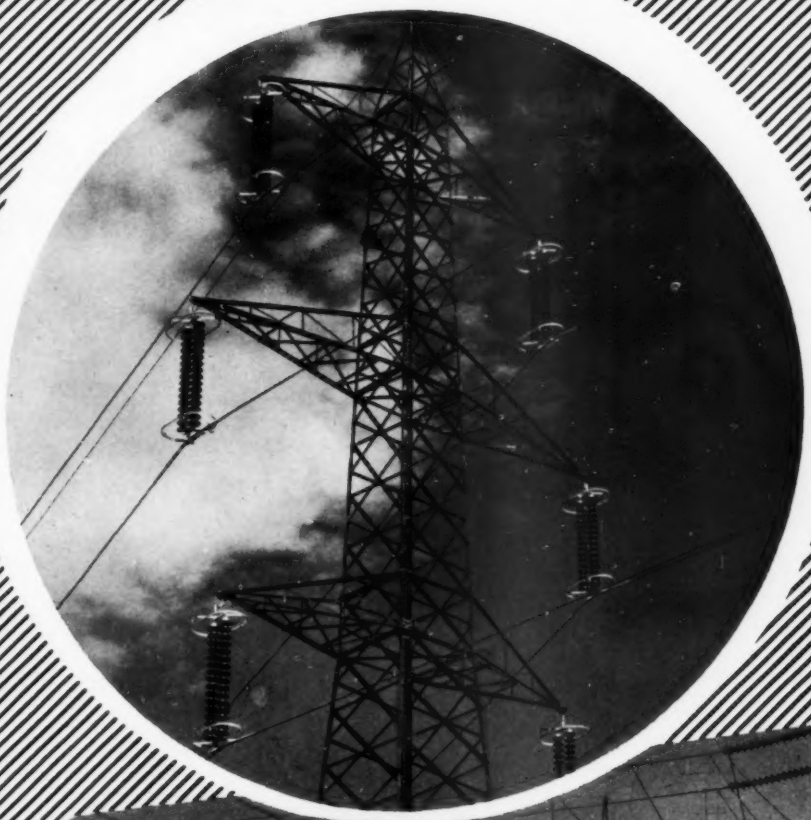
Above and at right:
Vertical, 4 nozzle Pelton type model turbine, with transparent plexiglass casing, being tested.

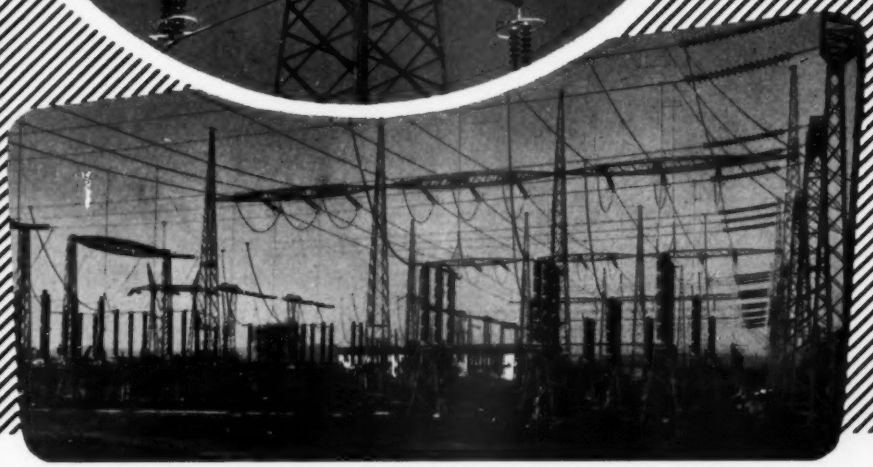




Three Kaplan runners of different specific speeds which have been tested

SALVI





For 30 years Messrs. A. Salvi and Company, Milano, Italy, have specialized in the design and manufacture of complete equipment for Overhead Transmission Lines, including Compression Dead End Assemblies and Joints.
The Works of the Salvi Company are engaged exclusively in the production of high grade fittings for High Voltage Transmission Systems, up to and including voltages of 220/380 KV, and for all sizes of Copper, A.C.S.R. and Aluminium Alloy Conductors.

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Kuljian has designed, engineered, and supervised construction of power plants the world over... with total electric generating capacity exceeding

2,000,000 KILOWATTS!

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Behind every Kuljian project lie the combined skills and experience of a complete staff of internationally recognized experts in every phase of water resource development. No matter where *your* next project might be, The Kuljian Corporation is fully prepared to undertake efficiently all phases of design, engineering, and construction supervision to help bring about the quick, economical solution to your specific problem.

We invite your inquiry for detailed information.

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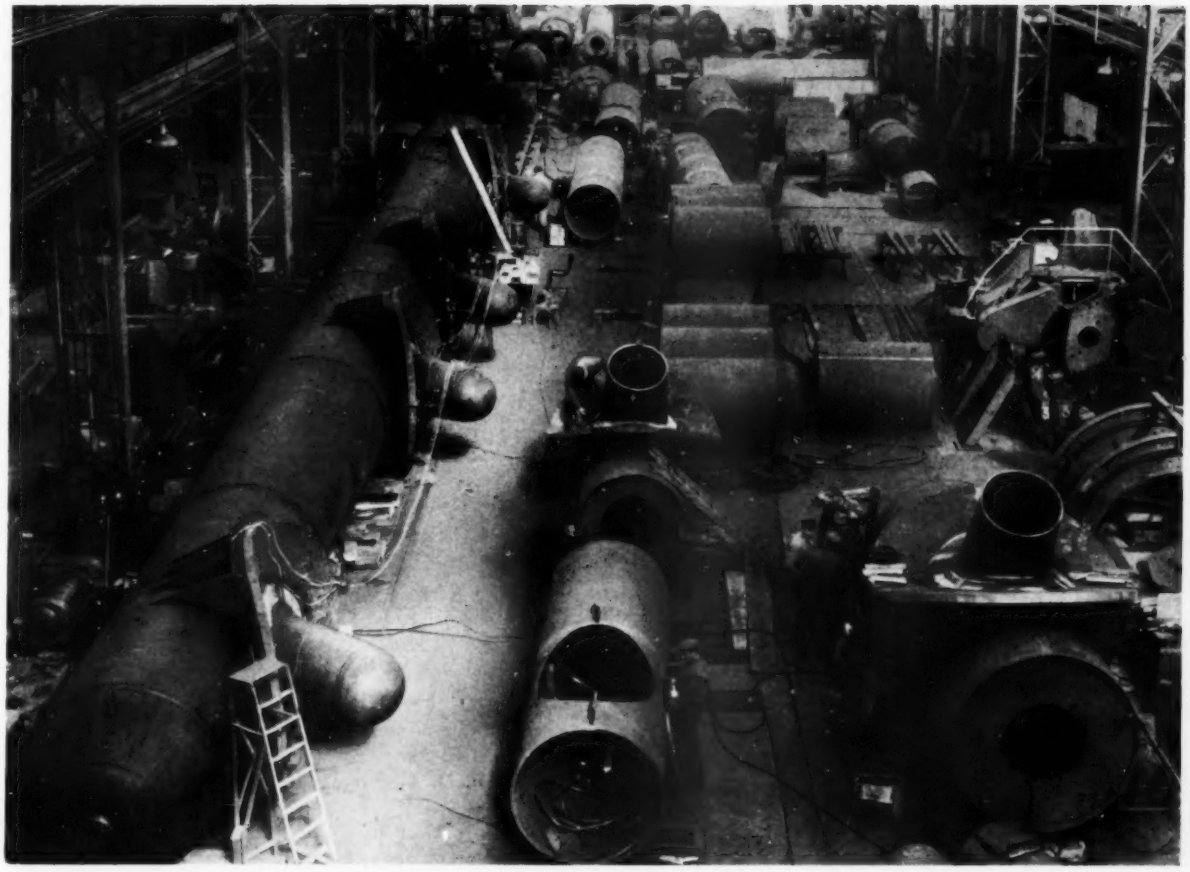


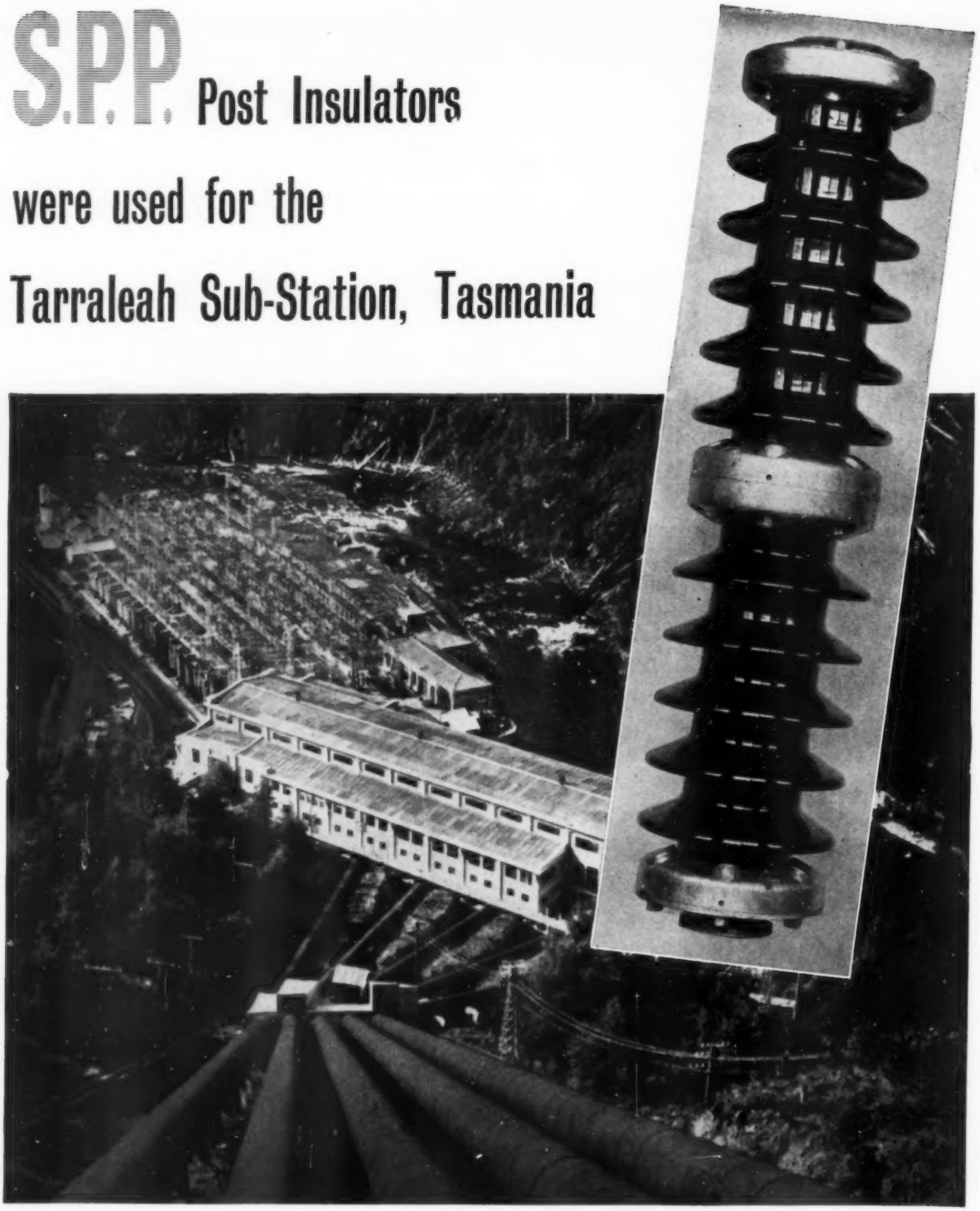
Downstream portion of the Grande Dixence Fionnay Power Station distribution pipe during tensometric shop tests. Max. wall thickness 52 mm., service pressure 96 at., test pressure 121 at.

GIOVANOLA **Monthey - Switzerland**

S.P.P. Post Insulators

were used for the

Tarraleah Sub-Station, Tasmania



The Sub-station at Tarraleah, Tasmania, is one of the many 110 kV. and 88 kV. sub-stations on the Hydro-Electric Commission's system equipped throughout with S.P.P. Cylindrical Post Insulators.

STEATITE & PORCELAIN PRODUCTS LTD.

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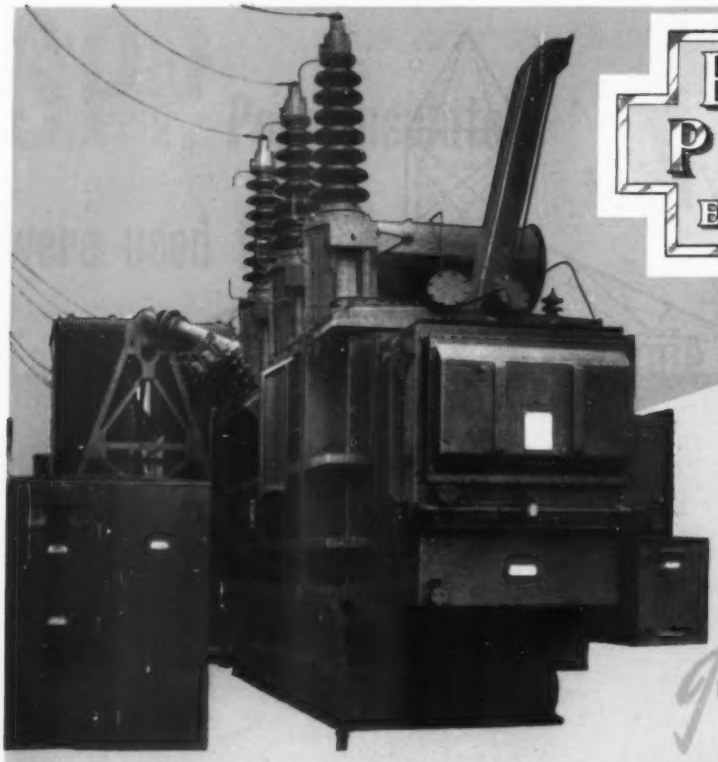

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EDINBURGH

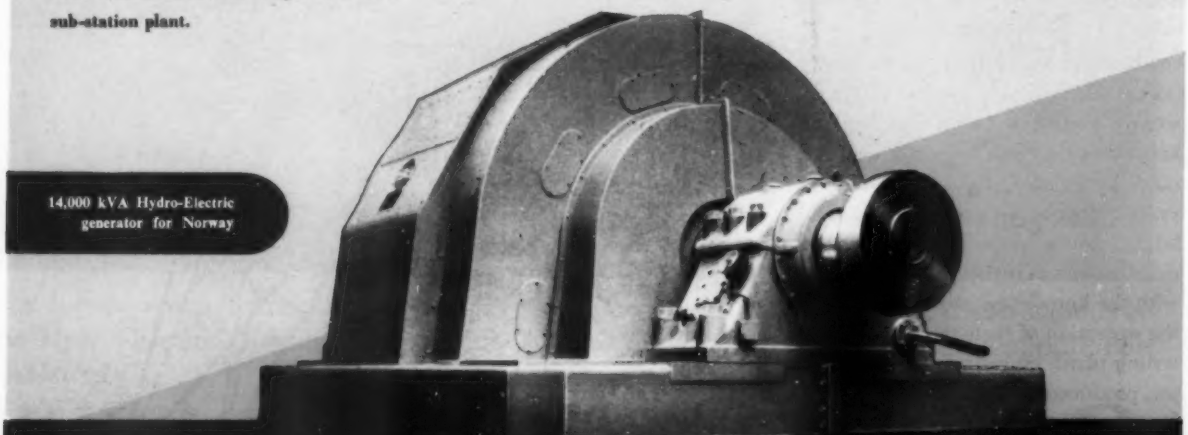
45 MVA 132/33 kV transformer for the Central Electricity Authority

Pioneers

OF THE ELECTRICAL INDUSTRY WITH OVER 50 YEARS' MANUFACTURING EXPERIENCE.

Activities embrace a wide variety of manufacture ranging from heavy rotating plant for the generation and conversion of electrical energy to motors for every sphere of industry: transformers of well over 100 MVA for 275 kV service to the smallest unit for distribution schemes and rural electrification; and complete rectifier sub-station plant.

For generations the name of Bruce Peebles has been famous for the manufacture of electrical power plant. Throughout its whole history the Company has ever been progressive, and its pioneering work has played an important role in the evolution of the present-day electrical age.



14,000 kVA Hydro-Electric generator for Norway

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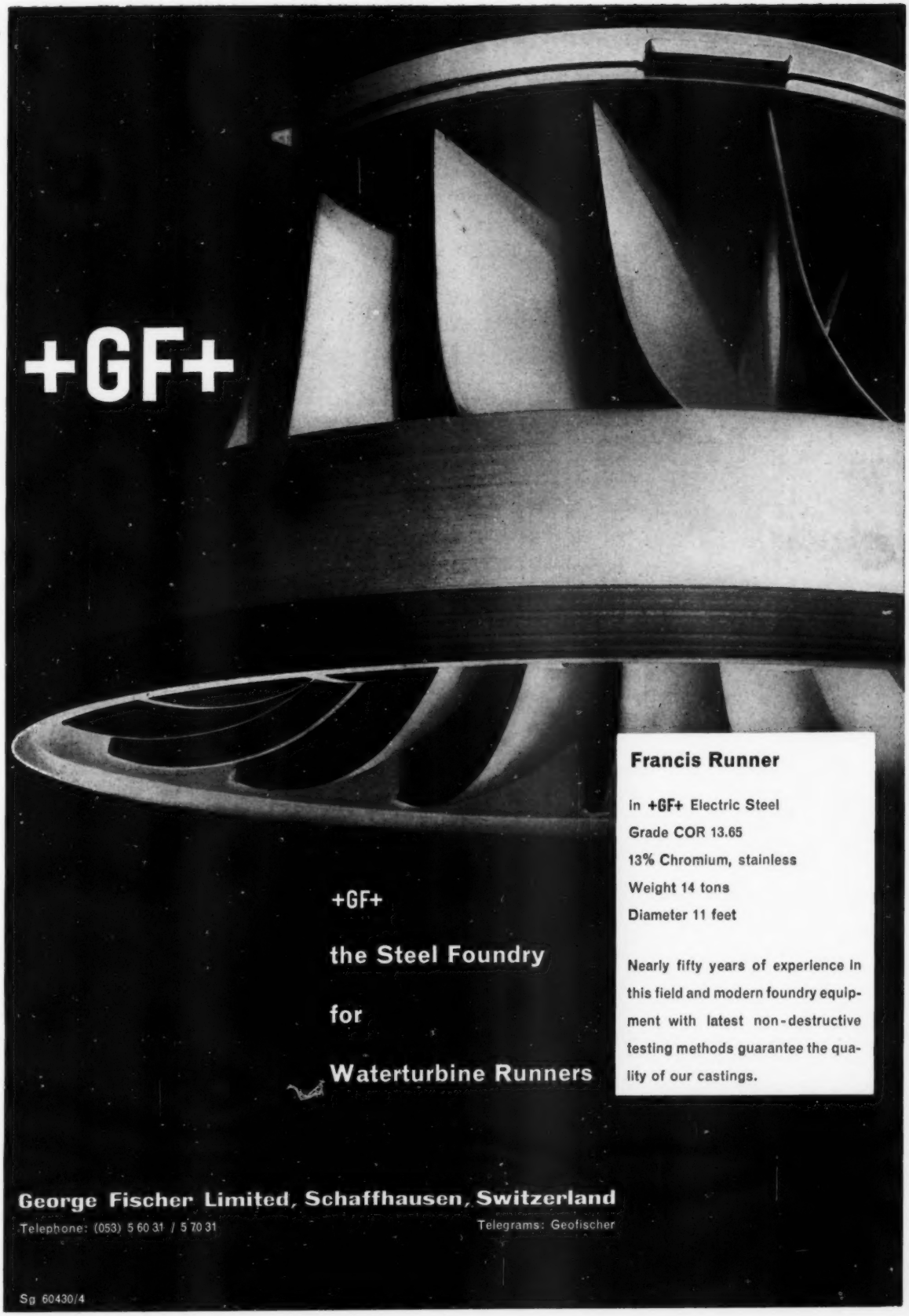
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275 kV

oil-filled cables

in Great Britain

at

Drakelow Power Station

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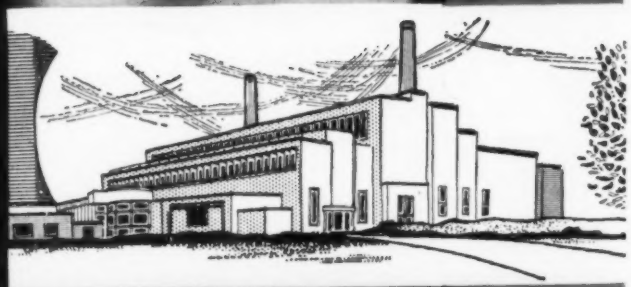
Pirelli-General 275 kV oil-filled cables have passed type approval tests on all conductor sizes up to 1·0 sq. in.

Consulting Engineers Merz & McLellan



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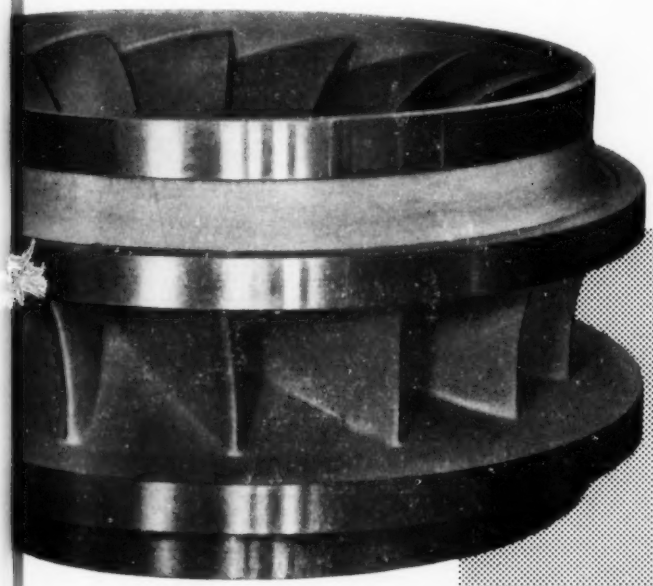


Turbines

governors

penstocks

valves

travelling cranes

Ateliers de Constructions Mécaniques de Vevey S.A.

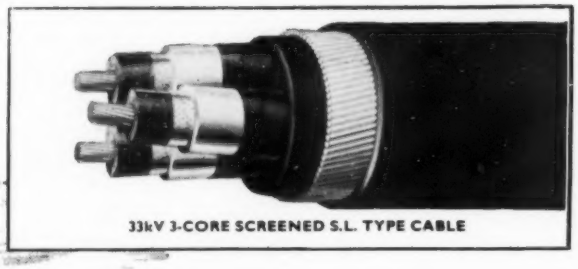
1471 E

POWER

Created by Bernini in the 17th century, this lithe sculpture of David stands in the Galleria della Villa Borghese, Rome, a gleaming tribute in bronze to the sculptor's conception of power.

In our modern world, power — electrical power—is the great motivator of our industries, the light and warmth of our homes. And wherever electricity comes from, wherever it goes, Henley cables carry it surely and safely. No manufacturer has more experience in the design and manufacture of supertension cables up to 132 kV.

David, *by Bernini.*
In the Galleria della Villa Borghese, Rome.



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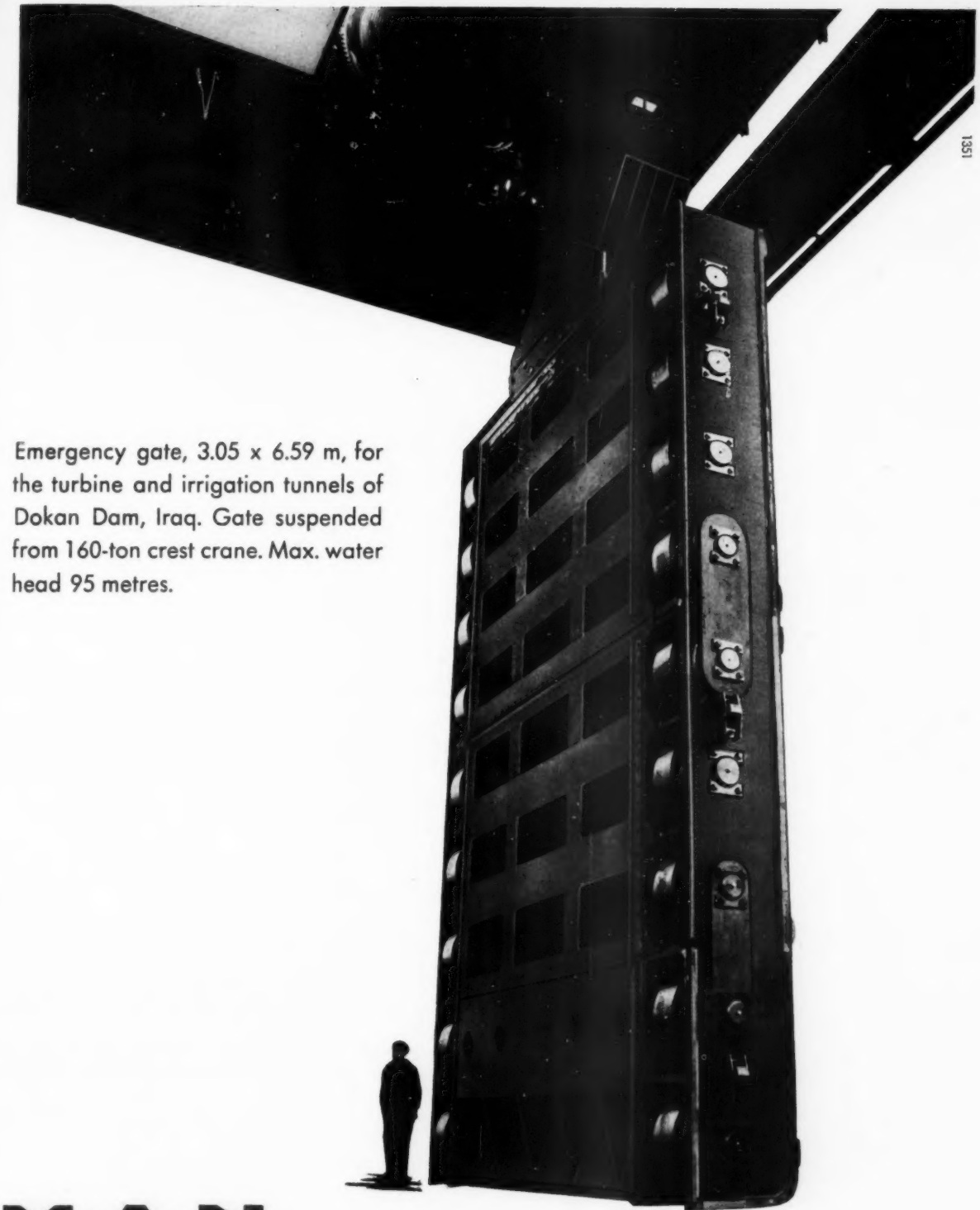
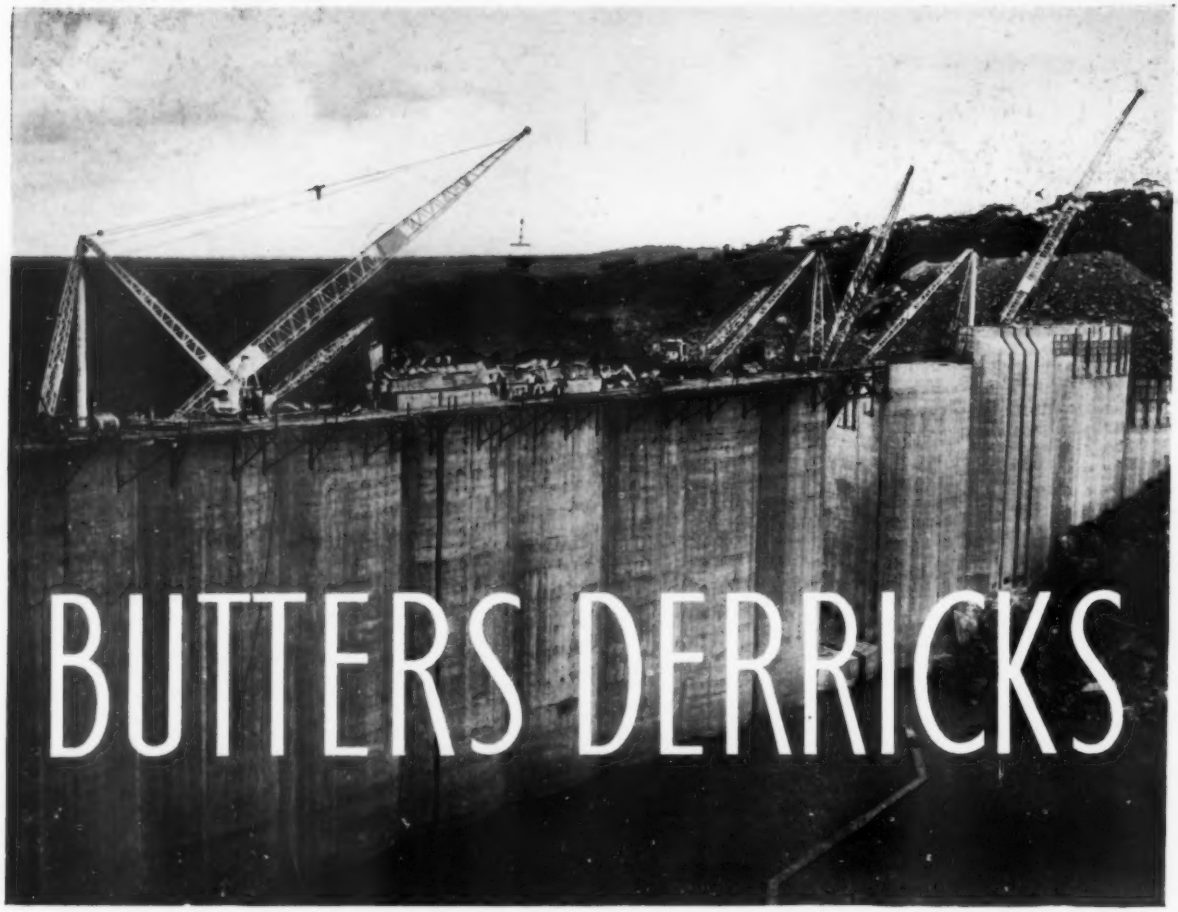


1351

Emergency gate, 3.05 x 6.59 m, for the turbine and irrigation tunnels of Dokan Dam, Iraq. Gate suspended from 160-ton crest crane. Max. water head 95 metres.

M·A·N
HYDRAULIC STRUCTURES

MASCHINENFABRIK AUGSBURG-NÜRNBERG AG GUSTAVSBURG WORKS
GUSTAVSBURG/HESSEN (GERMANY) REPRESENTATIVES IN ALL COUNTRIES



The Dam Builders

These pictures show several 7 ton Butters Derrick cranes in use on the construction of a dam in Rhodesia.

The difficult conditions and terrain often encountered in such work demand a very high degree of efficiency in the plant and ability to keep going with minimum maintenance. Butters Derricks have proved ideal—on dam construction all over the world, from New Zealand to the North of Scotland, from Iraq to Eire. Electric, Steam or Diesel drive can be provided in sizes to suit every purpose.

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For Harrsele Power Plant in Sweden, KMW have on order three Francis turbines, each developing 88,000 H.P. at 173 feet net head and 125 r.p.m. The first unit was put into operation in September, 1957.

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The contract awarded to Ferranti Ltd. by the Federal Power Board for the great Kariba Hydro-Electric Scheme on the Zambesi River covers two 120,000 kVA 330/234 kV 3-phase auto transformers with separate boosters; eight 60,000 kVA 330/88 kV 3-phase double-wound transformers; and two 60,000 kVA 330/33 kV 3-phase double-wound transformers. All the transformers will be provided with on-load tap changing gear. The Kariba contract is the largest overseas transformer contract to be placed with a British manufacturer, and emphasises the world-wide confidence in Ferranti Ltd. as builders of fine transformers.

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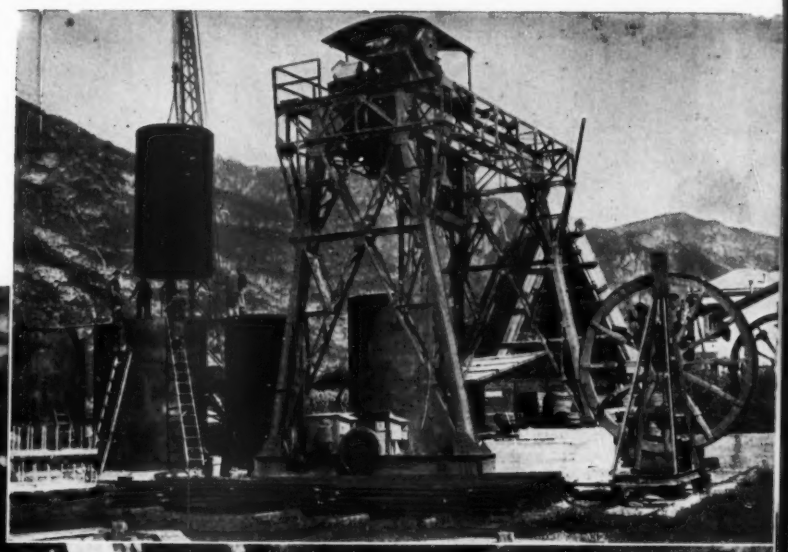

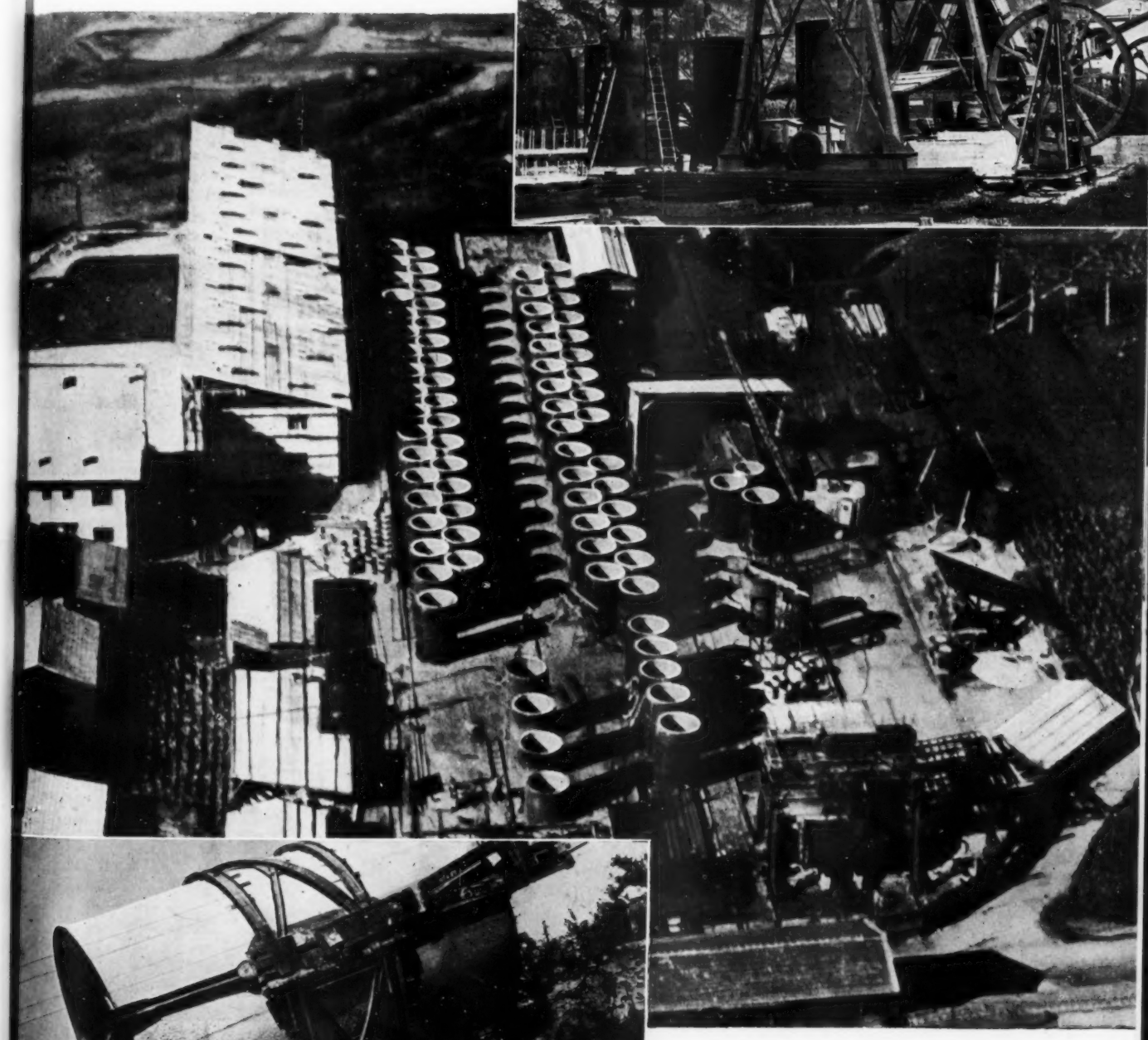

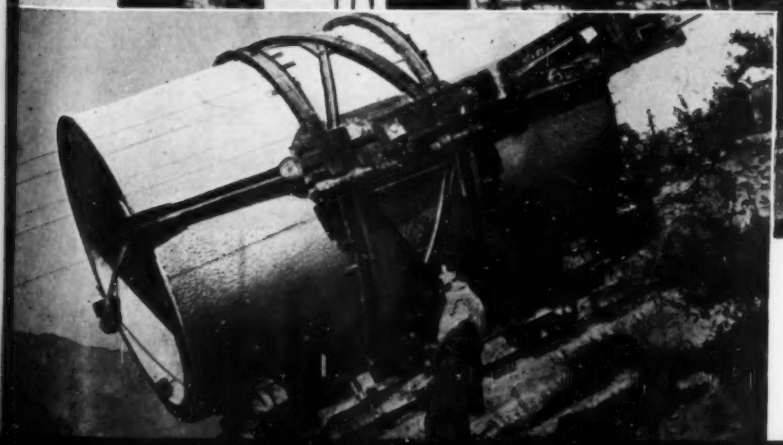

Main power penstock of prestressed reinforced concrete made for S.A.D.E. Soverzene Station and in service since 1950.

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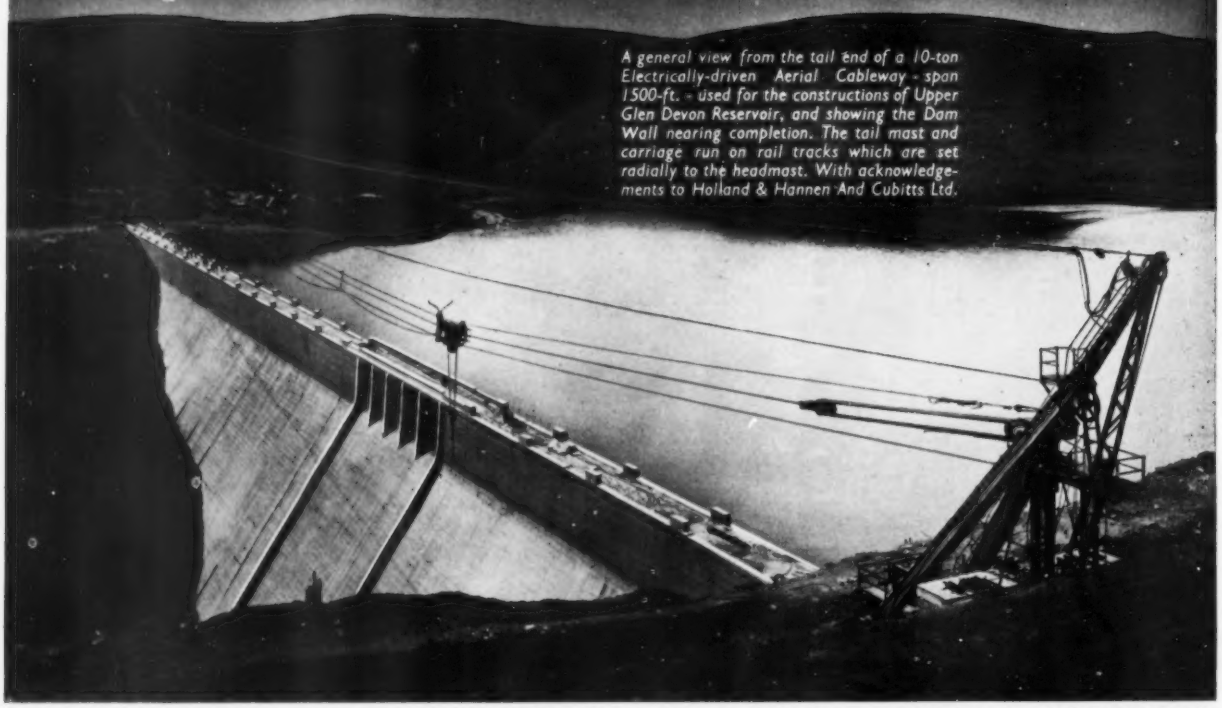
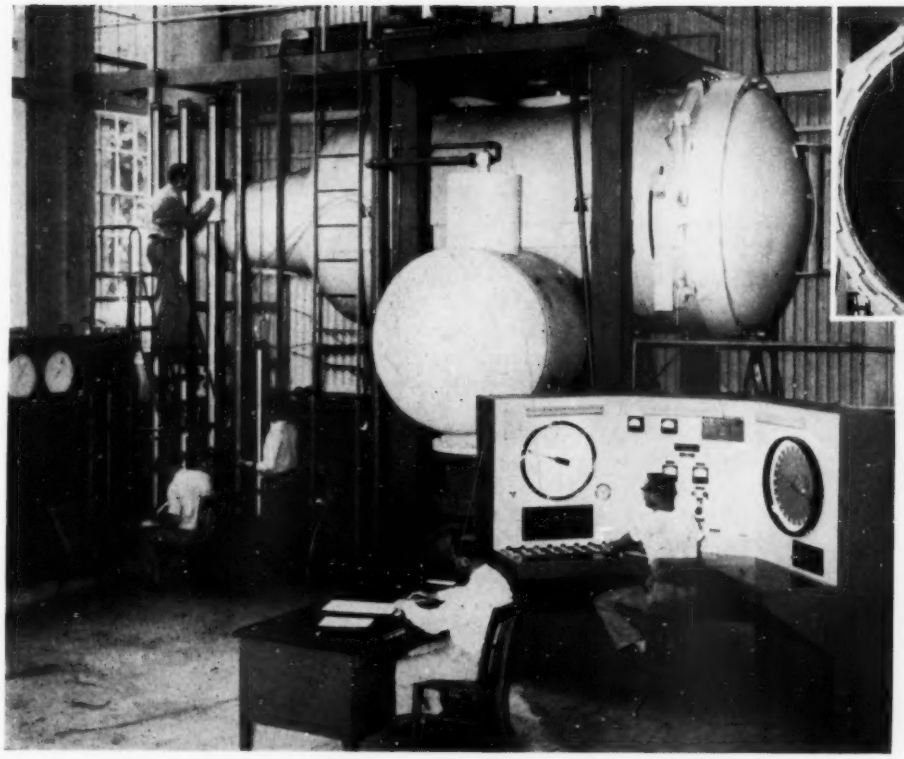
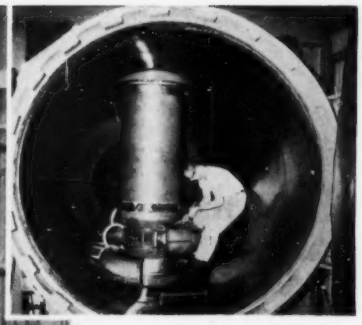


A general view from the tail end of a 10-ton Electrically-driven Aerial Cableway - span 1500-ft. - used for the constructions of Upper Glen Devon Reservoir, and showing the Dam Wall nearing completion. The tail mast and carriage run on rail tracks which are set radially to the headmast. With acknowledgements to Holland & Hannen And Cubitts Ltd.

JOHN M HENDERSON AND CO LIMITED KINGS WORKS ABERDEEN





A complete model turbine wheel case assembly, built to extreme accuracy, is shown being mounted in the testing chamber.

New high head test stand in the SMS Hydraulic Laboratory, including control panel and gages.

High Head Turbine Test Stand At SMS Laboratory DUPLICATES FIELD CONDITIONS

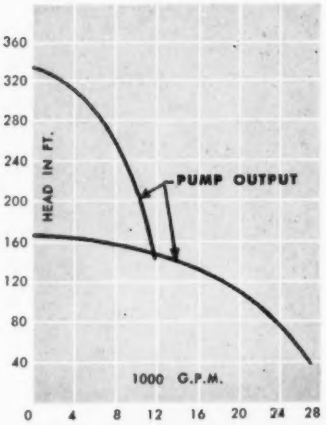
With the addition of its new high head test stand, the SMS Hydraulic Laboratory can now test homologous model turbines under controlled conditions that *duplicate* those found in the prototype installations. This is a substantial stride forward in hydraulic turbine research, since the complete reproduction of actual field conditions has not previously been possible.

The outstanding features of this new facility are:

- World's highest artificial testing head.
- Flows of 5,000 g.p.m. under 300-foot head, and 21,000 g.p.m. under 100-foot head.
- 300 H.P., 0 to 6,000 R.P.M., eddy current dynamometer.
- 100-ton water cooling system to maintain prototype water temperature.

Both basic research and formal contract acceptance tests are performed here. Test stand equipment permits testing of all Kaplan and Propeller, as well as many Francis turbine designs, under conditions of *head* and sigma that *duplicate* those found in the field. This provides a high degree of testing accuracy and permits accurate testing of settings that cannot be satisfactorily field tested.

For further information on the significance of this high head test stand, write to S. Morgan Smith Co., York, Pa.



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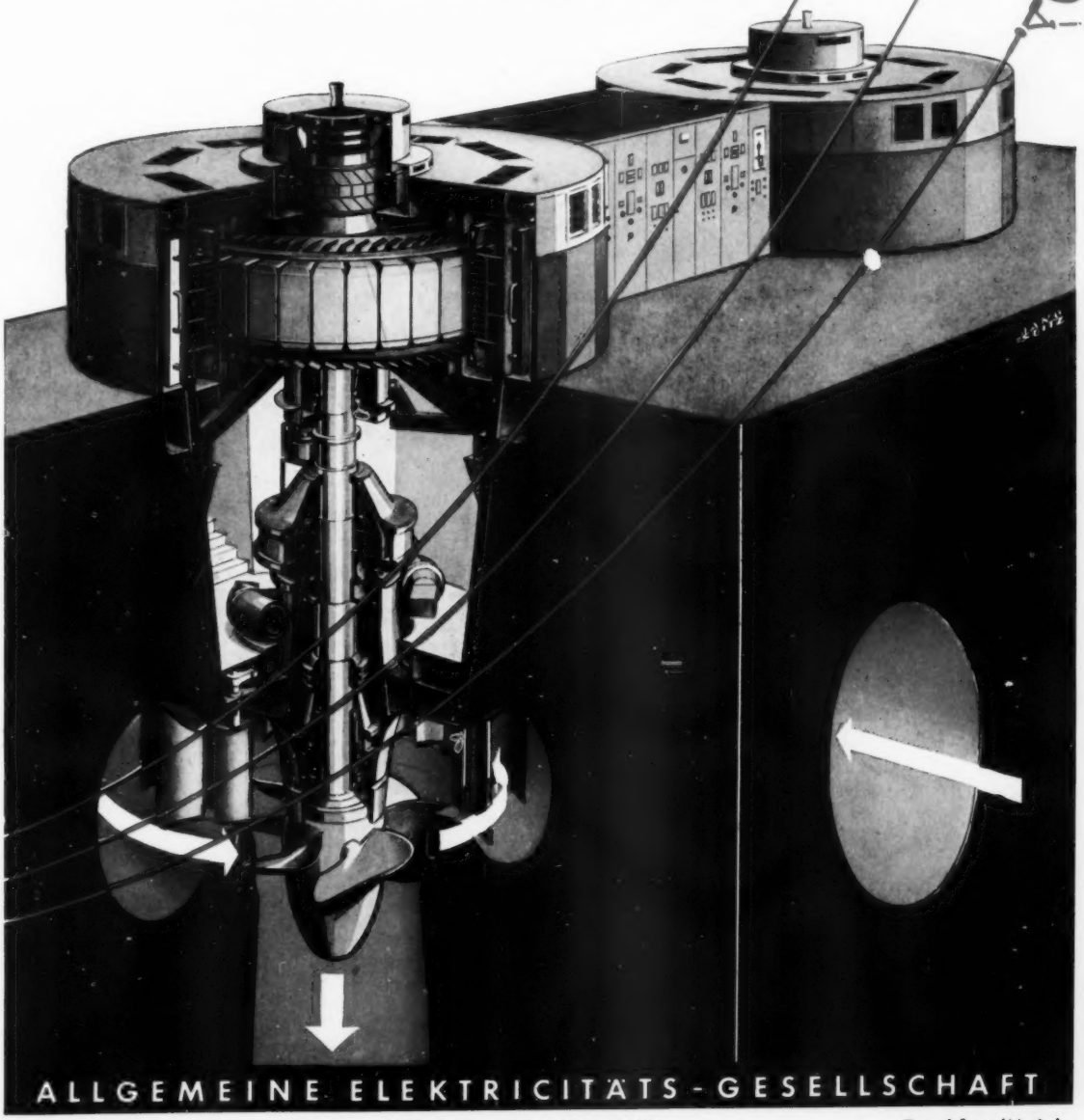

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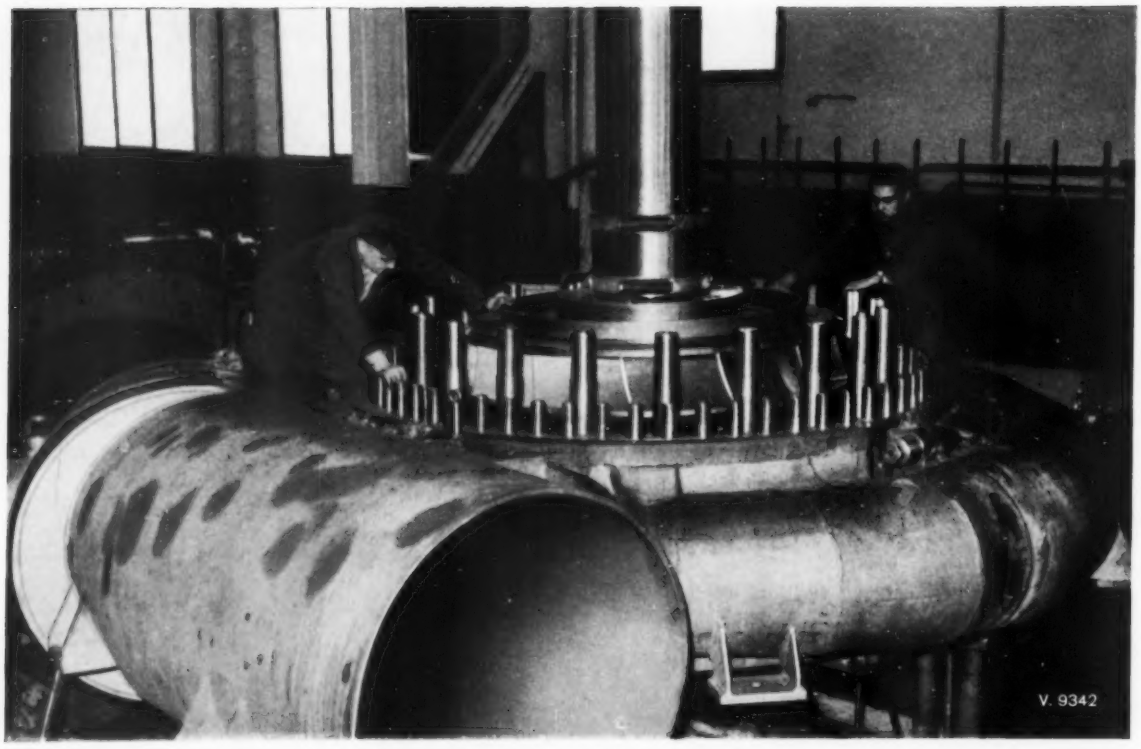

One of three 50,000 BHP Voith Francis Spiral Turbines for the Periyar Power Station in India. Net head 1,245 ft.

VERTICAL-SHAFT FRANCIS SPIRAL TURBINES

Supplied to	Number	h.p. each	Head (feet)
Barázar (Spain)	2	48,500	1,111
Ixtapantongo (Mexico)	1	76,000	1,020
El Cóbano (Mexico)	1	40,700	1,080
Saucelle (Spain)	4	86,500	203
Hirakud I (India)	2	34,500	108
Caniçada (Portugal)	2	38,300	321
Bouça (Portugal)	2	34,000	184
Seyhan (Turkey)	2	32,000	121
Mocorongo (Colombia)	1	42,000	1,000
Under construction:			
Canelles (Spain)	3	50,000	413
Schwarzach (Austria)	3	42,000	472
Macagua (Venezuela)	4	87,200	148
Periyar (India)	3	50,000	1,245
Machkund (India)	3	36,000	836
Gandhi Sagar (India)	3	34,000	149

Spiral casing either completely shop-welded or pre-fabricated for welding on site. Shaft guided in self-lubricating bearing. Guide vanes operated mostly by Voith annular servomotors. Draft tube liner of plate steel. Runner of cast steel, cast steel with stainless steel facing, or of chrome cast steel. Write for Voith descriptive literature.

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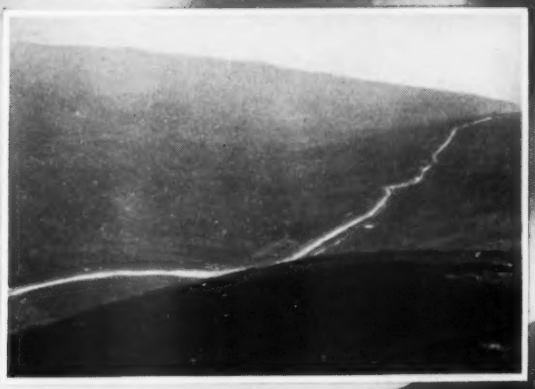

The problem varies from site to site—here are some 40 miles of access roads built complete with bridges by The Mitchell Construction Company in Glen Almond, Glen Tarken and Glen Lednock for development of the St. Fillans Section of the North of Scotland Hydro-Electric Board's Breadalbane Scheme.

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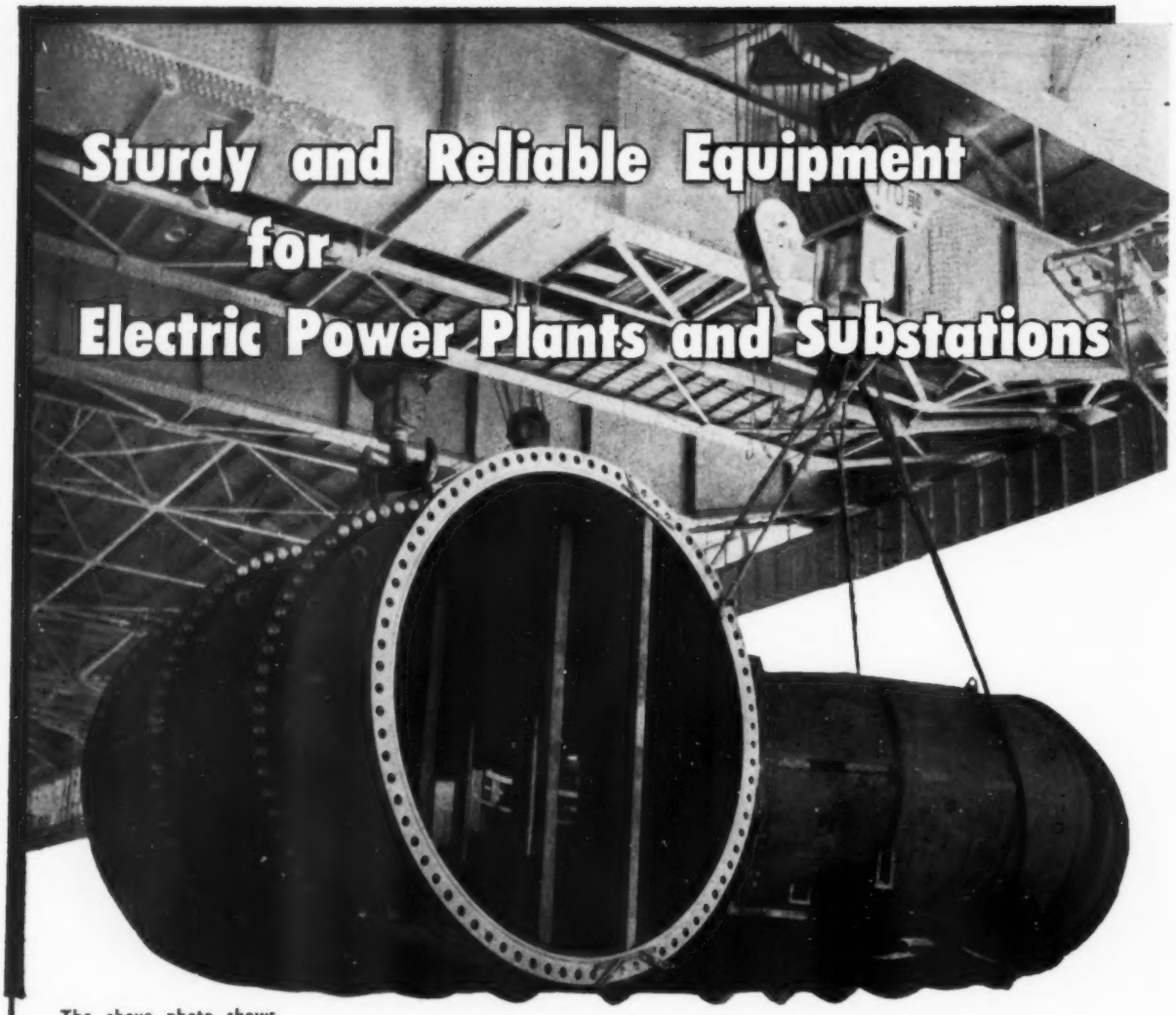

The above photo shows Casing for 150,000 HP Francis Turbine to be installed in the Bahkra Power Plant, India.

HITACHI's alternators are backed by more than fifty years of indefatigable research and engineering experience. Several thousand HITACHI alternators in a wide variety of capacities have already been supplied with satisfactory results to power plants both at home and abroad.

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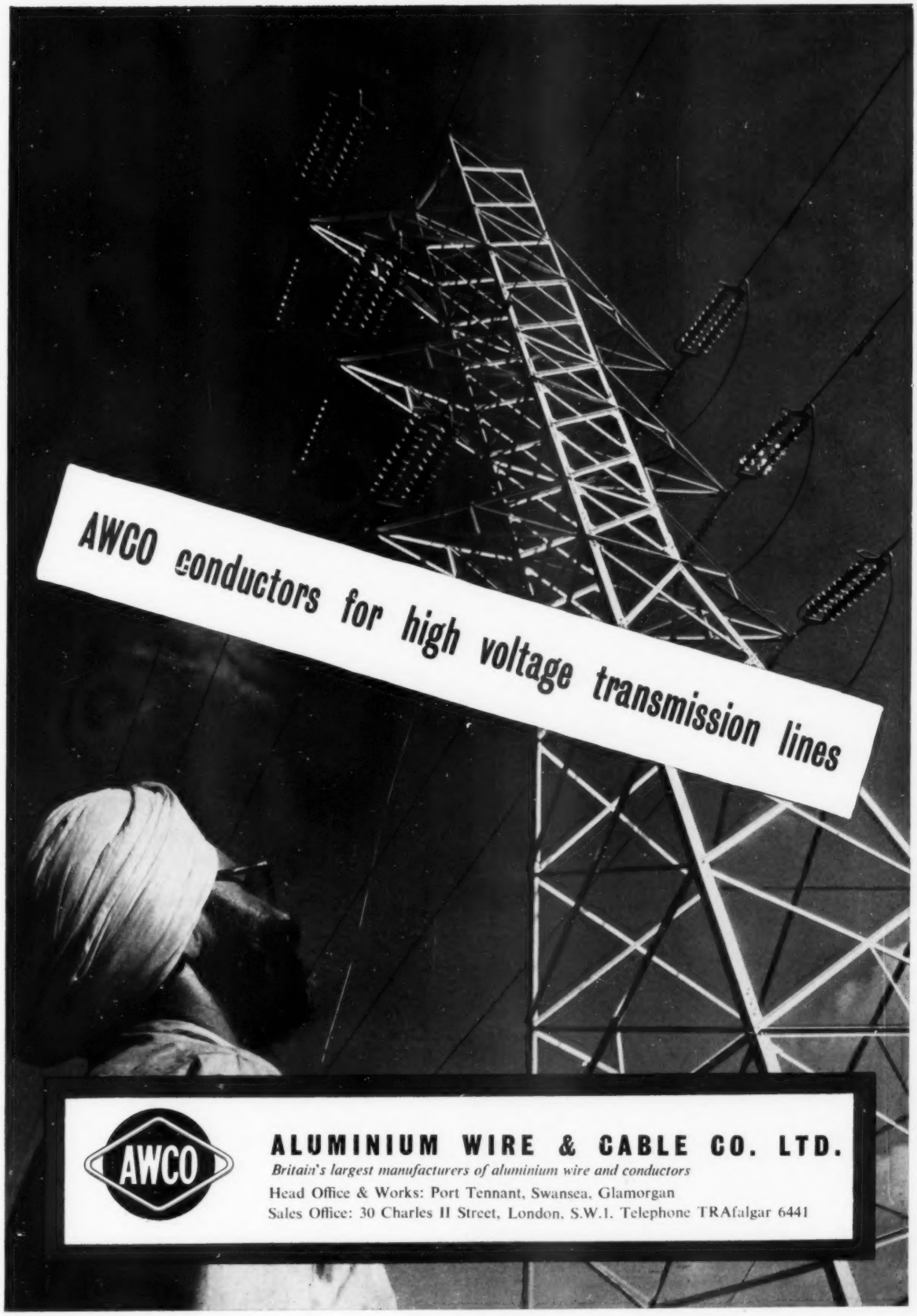
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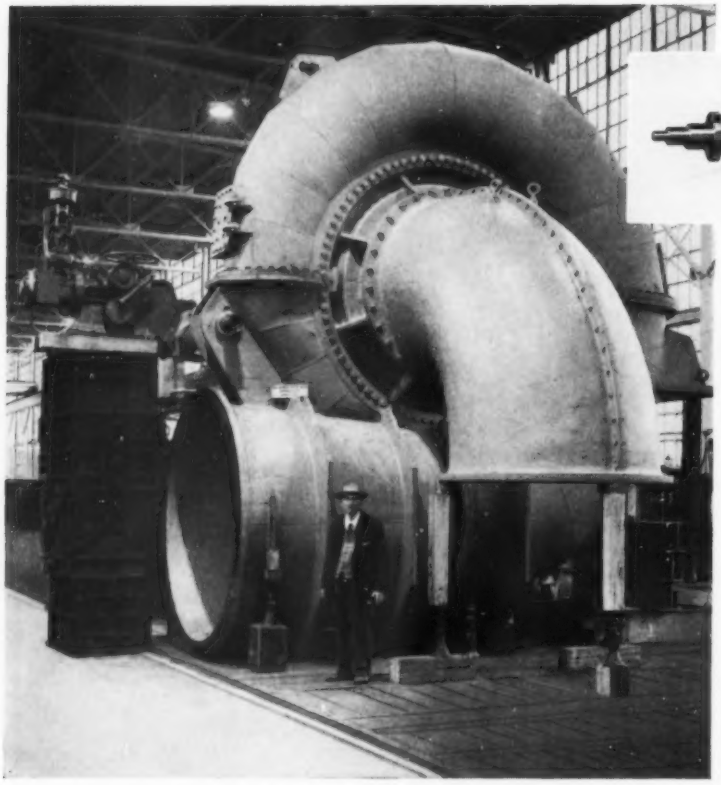
The low net head of 68 ft. at the Pleasant Valley Power Plant near Los Angeles required the design and construction by Leffel of a giant, horizontal spiral case turbine unit. This turbine, rated to develop 3,520 H.P. under the net head of 68 ft. at a speed of 257 R.P.M., drives a horizontal generator. A common shaft carries both the generator rotor and the stainless steel runner. A synchronous by-pass valve allows adjustment of flow for both turbine and by-pass discharge to meet water passage requirements. Photographs here show this giant turbine in various stages of production at the modern Leffel plant.



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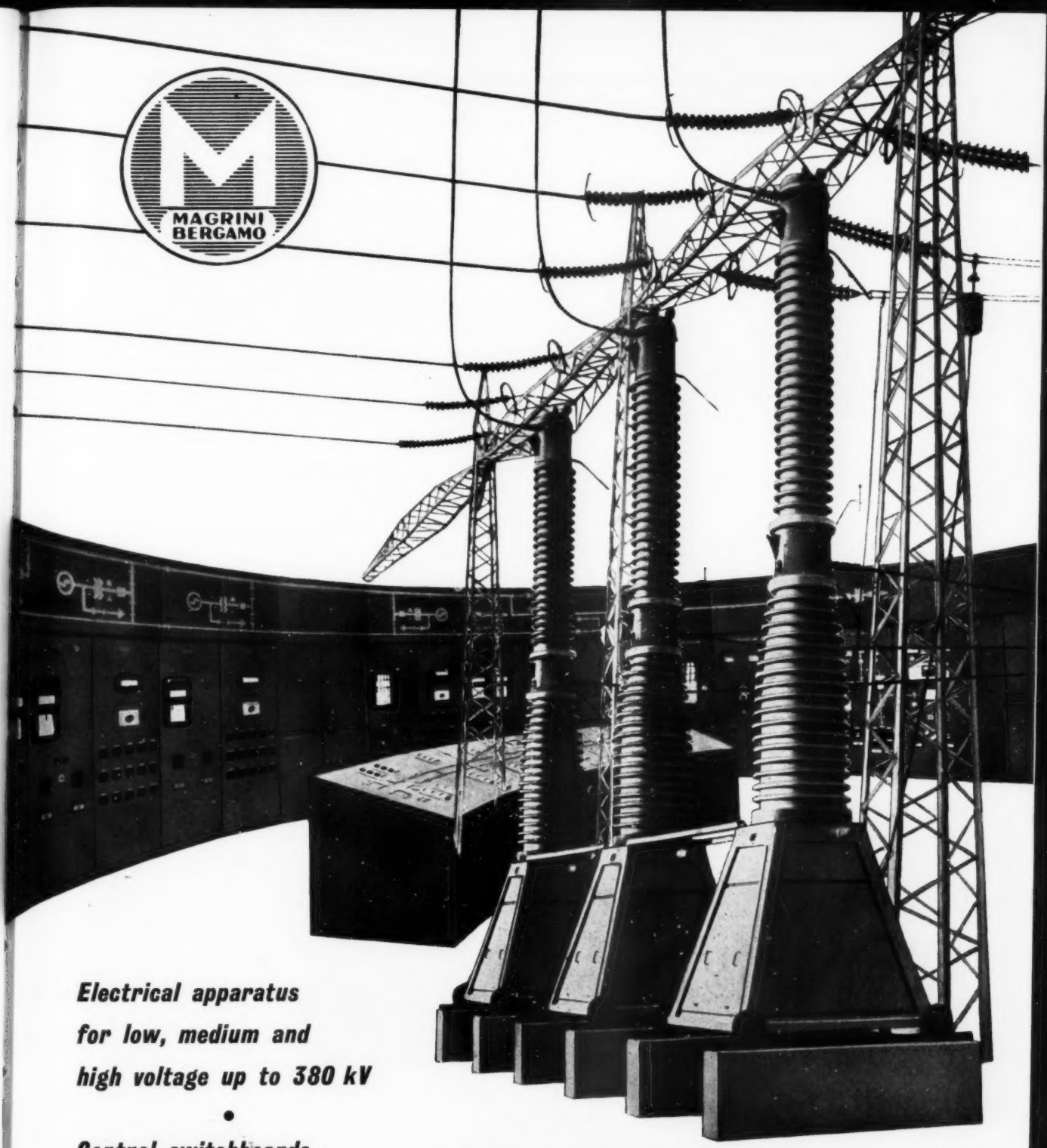

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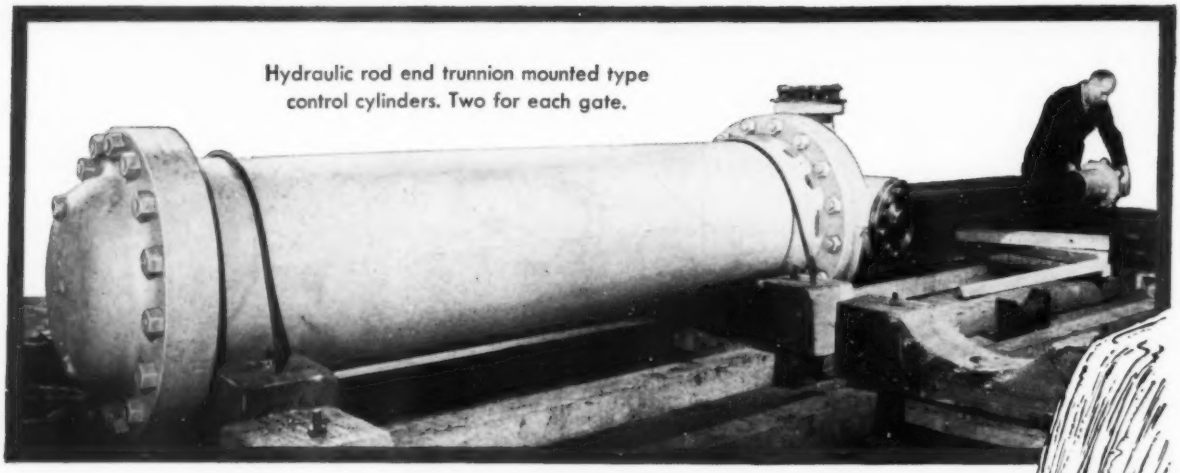


Hydraulic rod end trunnion mounted type control cylinders. Two for each gate.

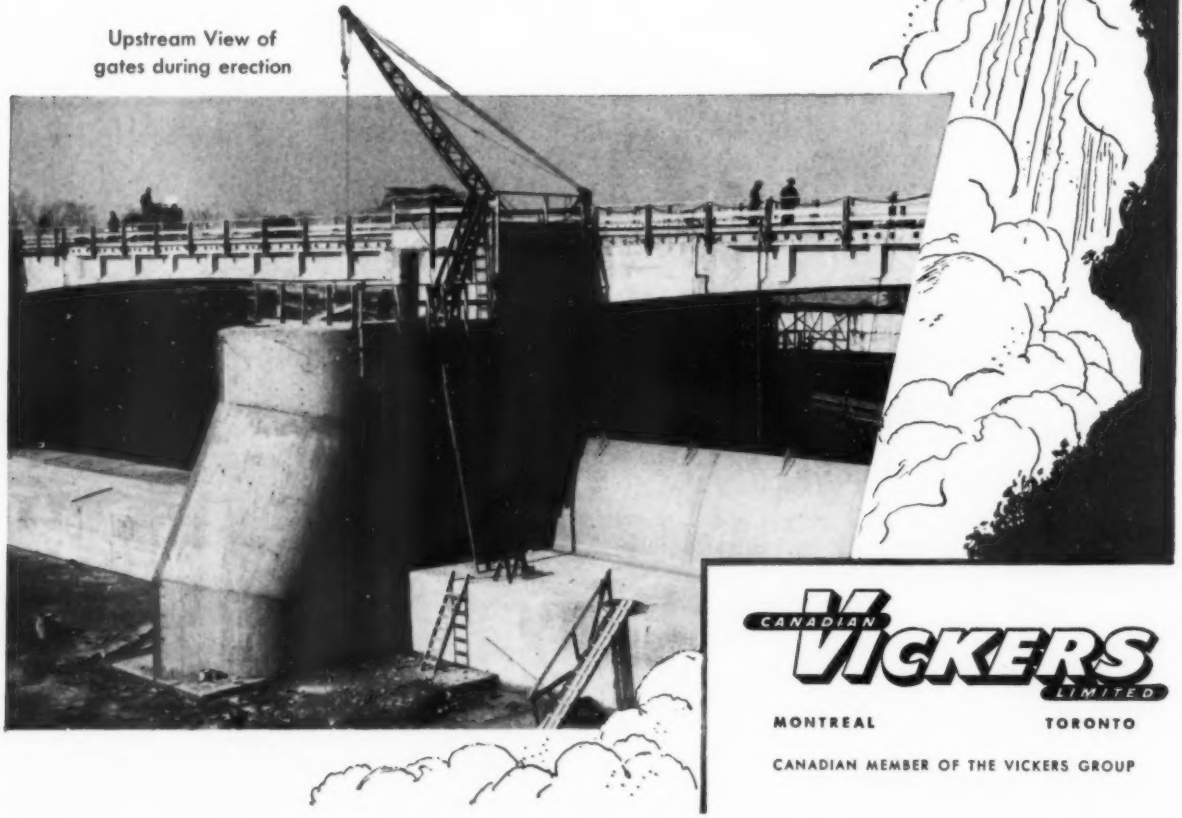
CONTROLLING THE FLOW OF NIAGARA

The flow of Niagara, one of Canada's greatest sources of electric power, is controlled by Vickers-built submersible gates of the "fishbelly" type.

Working with the M.A.N. Company, a Canadian Vickers associate, who designed the 100 ft. wide section welded gates, thirteen gates were supplied along with twenty-six servomotors and thirteen pumping sets which were entirely engineered by Canadian Vickers.

These gates are another example of the precise engineering that goes into all Vickers-built water control equipment.

Upstream View of gates during erection



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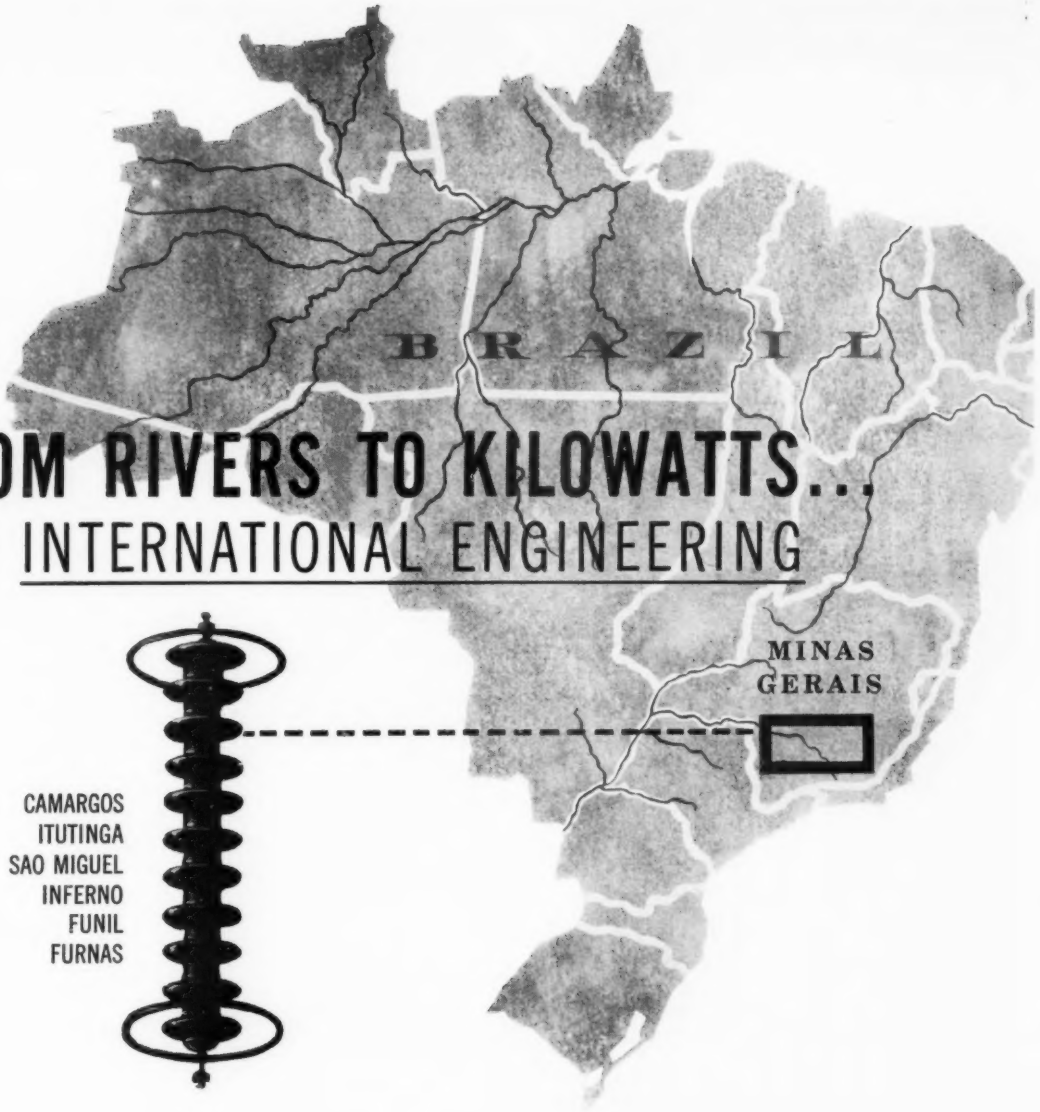

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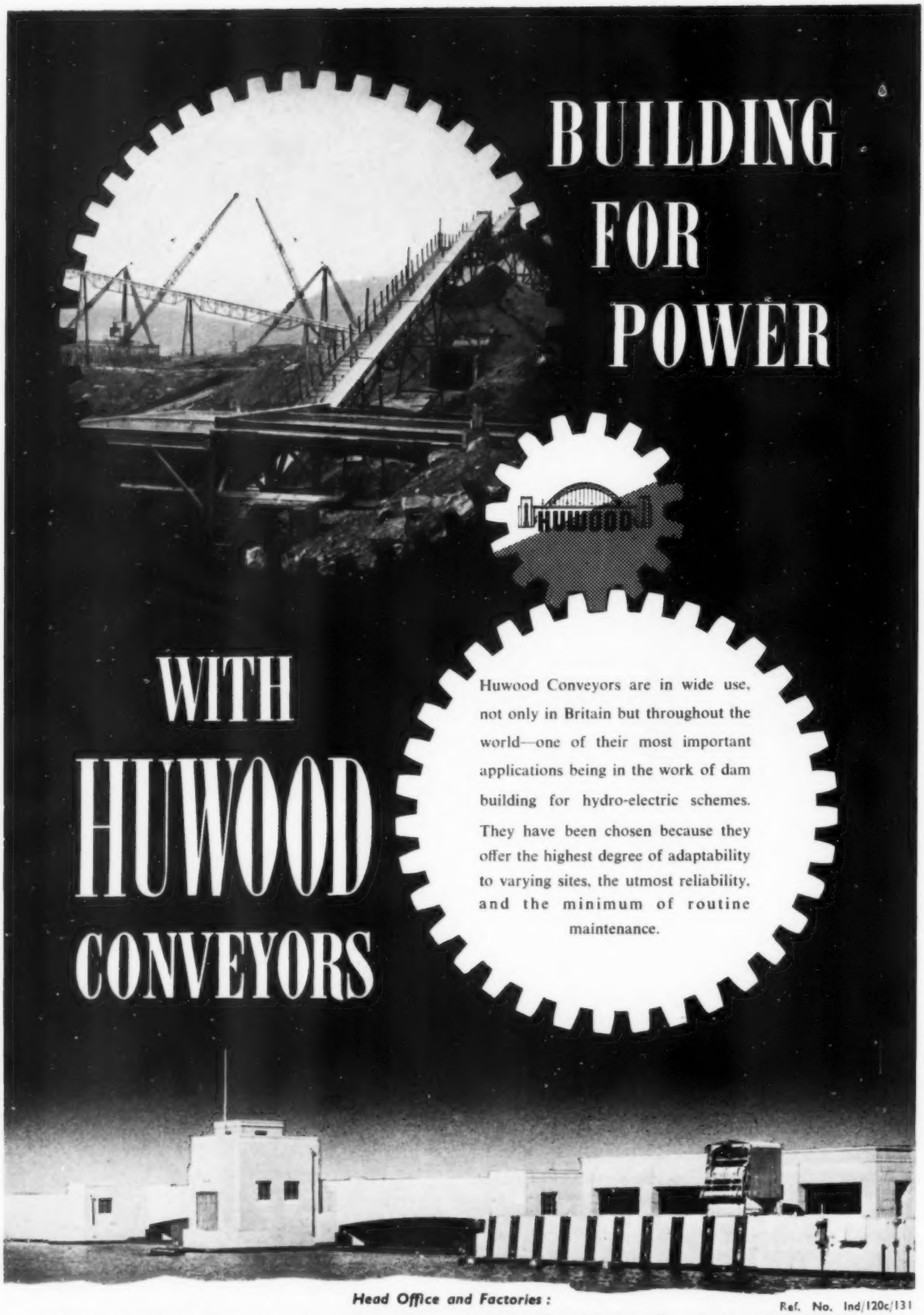
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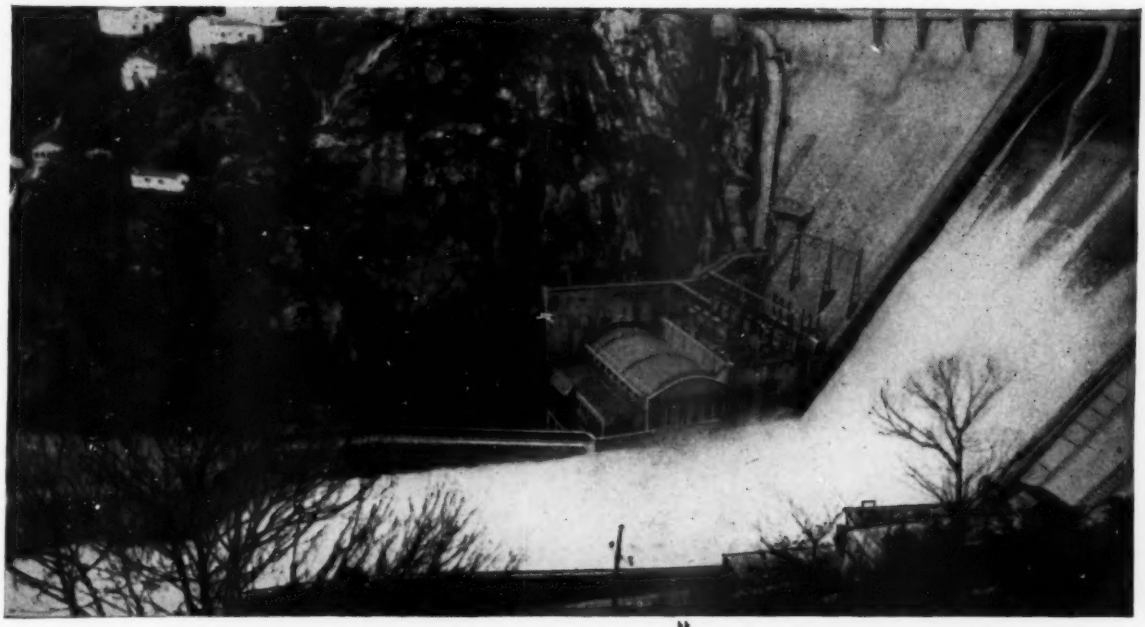
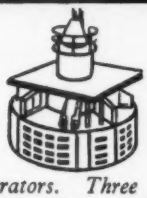
Los Peares Hydro-Electric Power Station

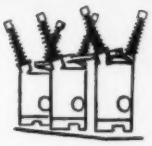
ANOTHER B T-H INSTALLATION



All over the world BTH hydro-electric equipment is helping to harness natural resources—helping to produce power and prosperity. In Spain, BTH (as main contractors) equipped completely the Los Peares hydro-electric power station.

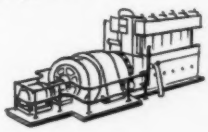


Generators. Three 62,400 kVA sets with Boving Francis-type turbines.



Main and auxiliary switch-gear; control and relay equipment.

Generator transformers (ten, 20,500 kVA single-phase units), and distribution transformers.



Power Station auxiliaries, including auxiliary diesel-generator.

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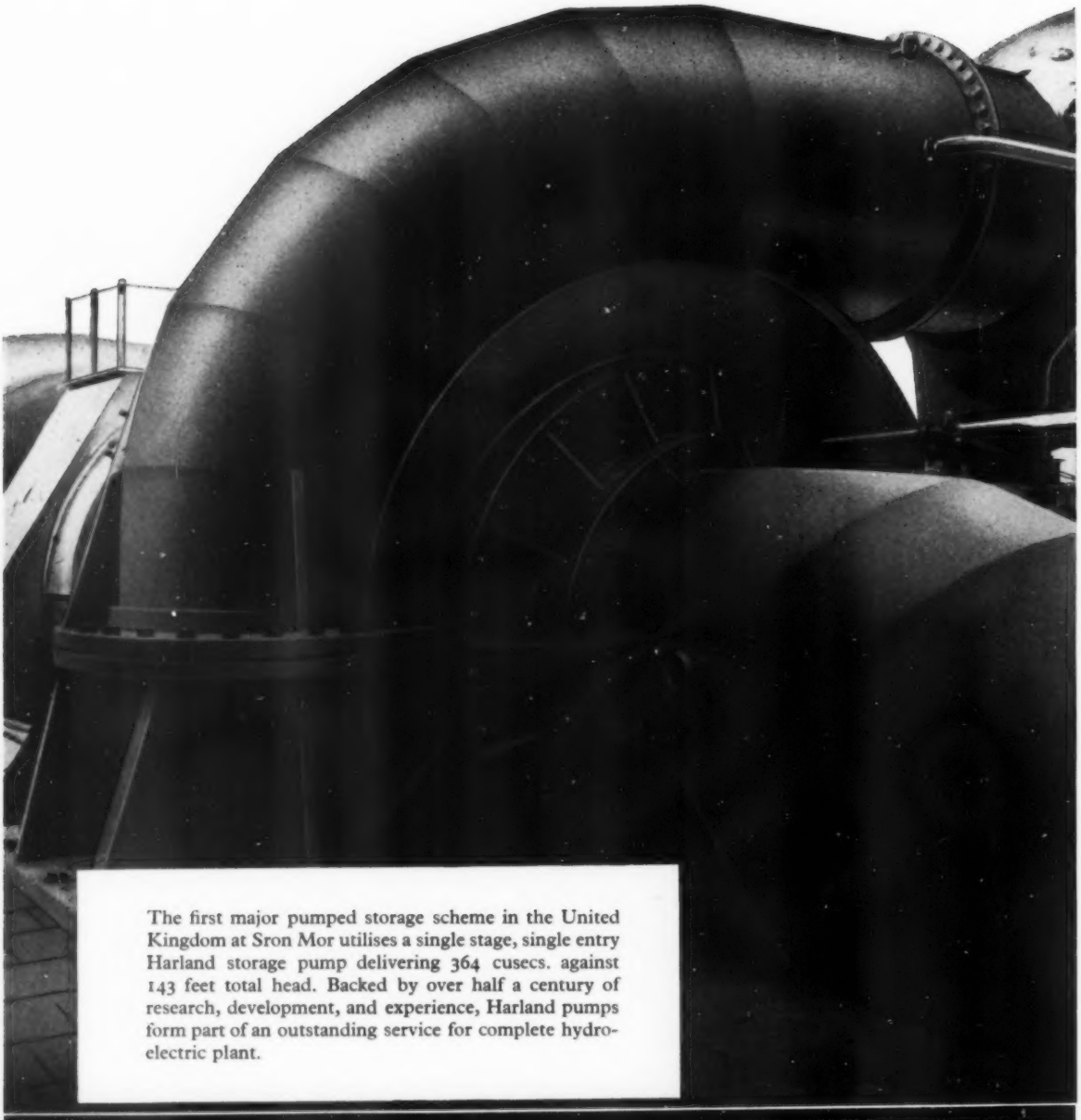
storage pump at Sron Mor



The first major pumped storage scheme in the United Kingdom at Sron Mor utilises a single stage, single entry Harland storage pump delivering 364 cusecs. against 143 feet total head. Backed by over half a century of research, development, and experience, Harland pumps form part of an outstanding service for complete hydro-electric plant.

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Editor: H. STOREY, M.C., B.Sc., M.I.M.E.

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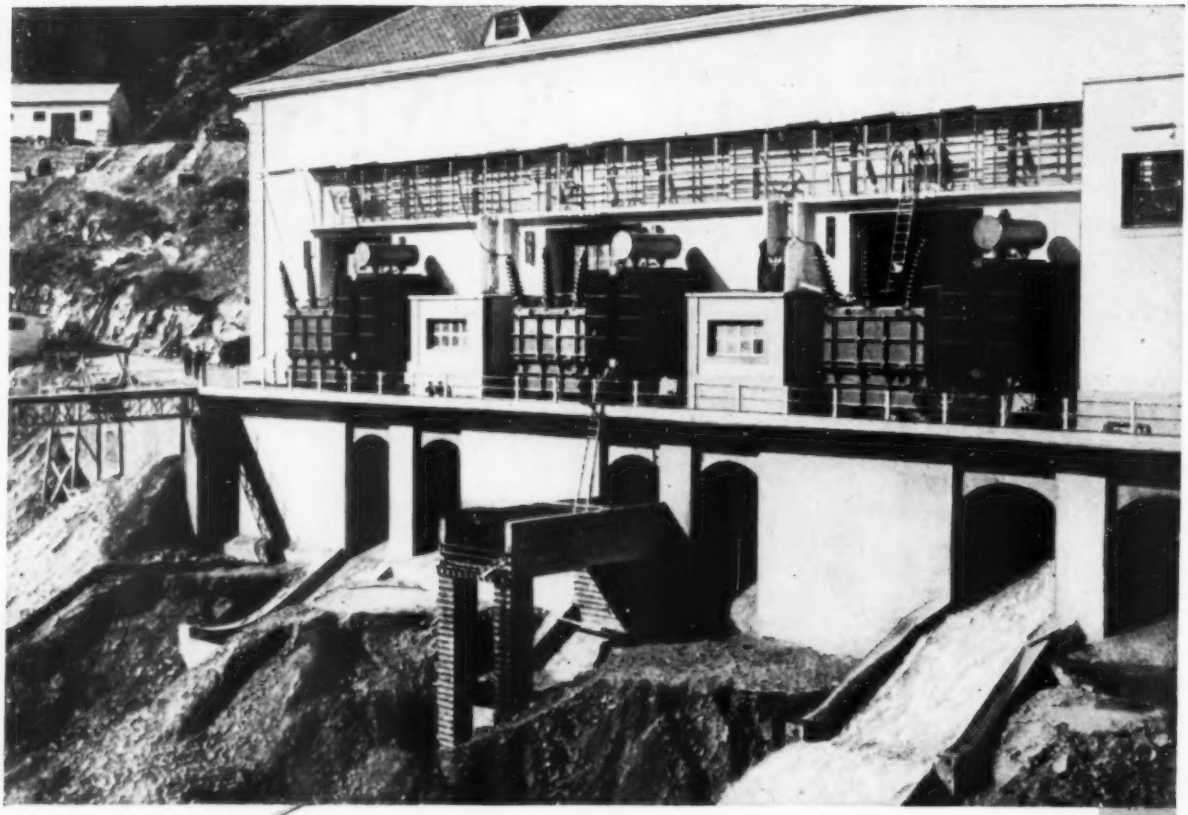
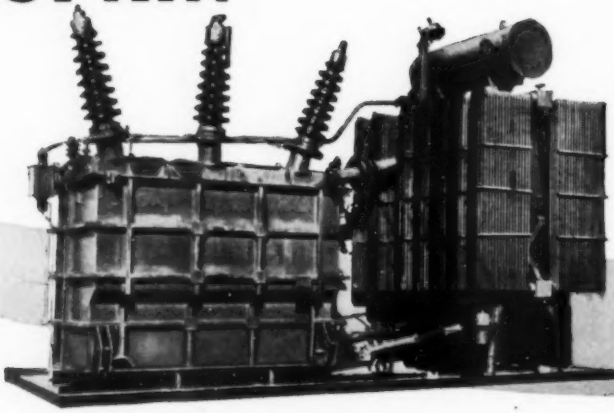


Three 16,000 kVA 6.6/138 kV Transformers at the Saltos del Sil Power Authority. The transformers were specially constructed to meet site requirements.

G.E.C.

TRANSFORMERS IN SPAIN



THE GENERAL ELECTRIC CO. LTD., MAGNET HOUSE, KINGSWAY, LONDON, W.C.2

WATER POWER

A technical journal devoted to the study of all aspects of Hydro-Electric Development

Tunnelling Records

DESPITE the drift in world thought towards the establishment of state-controlled or monopolistic structures, the competitive instinct remains in most of us, and, indeed, a healthy spirit of rivalry —provided it is kept healthy—can be of the essence of progress. Thus it is that the setting up of records seems to respond to a basic necessity of human nature, and it is not surprising that even in the hydro-electric field we are confronted with claims and counterclaims in respect of records of various kinds.

A few years ago, for instance, there was an intriguing little discussion in our columns as to which was the world's longest water tunnel, and more recently, more than one country has claimed the distinction of building the world's largest hydro-electric generating set.

Within the last two or three years, as our past issues reveal, there has been a spate of claims in regard to world tunnelling records, and our correspondence columns these last few months have testified to the interest that these claims have aroused.

To recapitulate these claims as far as we can collate them, the first of the present series appears to have been made in March 1955, when an advance of 428 ft. was made in seven working days through epidiorite in the 10½ ft. diameter Glen Almond tunnel in the St. Fillans section of the Breadalbane project of the North of Scotland Hydro-Electric Board. Conventional equipment was used comprising airleg-mounted drills, overthrowing shovel loaders and mine cars.

From a statement in July of the same year, this figure was raised at a different tunnel, Allt-na-Lairige, but again on a Scottish hydro project. On this occasion 444 ft. of 6½ ft. by 8 ft. tunnel were cut in a seven-working-day period. Although the drilling and mucking machines were standard, use was made of a special shuttle car that could accommodate a whole round of broken rock, and a high compressed-air pressure was furnished to the face machinery.

The record that now appears to be standing for the seven-day period was notified in December 1955, an advance of 557 ft. of 8 ft. 6 in. wide tunnel being achieved on the St. Fillans No. 4 tunnel. A detailed report of the driving methods and organisation on this scheme was published in our January 1956 issue.

These figures all relate to a seven-day period, but in October 1955 an advance of 402 ft. in six working days was notified for the Eucumbene-Tumut 24 ft. diameter tunnel in the Snowy Mountains scheme, and recently the maximum six-day advance in this tunnel was raised to 484 ft. On an arithmetical extrapolation this corresponds to a seven-day advance of 565 ft., and it is doubtless on this basis that it has been claimed as a world record.

Other claims have been based on the monthly rate of advance, it being held that this gives a truer index of sustained capacity. Thus, as we recorded last August, the contractors for the North of Scotland Hydro-Electric Board's Lednock tunnel claimed an advance of 1,621 ft. in an 8 ft. diameter tunnel in a 30-day period based on a working week of 134 hours. This record was challenged in a letter in our October issue pointing out that on the St. Fillans section of the Breadalbane scheme a 30-day advance of 1,837 ft. was achieved in a tunnel of 8½ ft. bed width. More recently, we have been notified of a claim to the 30-day world record set up in October 1956 by a Czechoslovak crew, which is stated to have advanced 1021·3 m. (3227·3 ft.) in a single tunnel face of 9 sq. m. (83·61 sq. ft.) section.

A significant point about all these claims is that they are based on rates of advance irrespective of the cross section of the tunnel. This gives point to the Swedish claim in respect of the Harrsele tunnel, 49 ft. wide by 59 ft. high, that the recorded advance in one month of 585 ft., presumably on two faces, constituted a world record "for a tunnel of this size." Perhaps a truer measure of the actual work involved is the amount of spoil removed; thus the Swedish State Power Board states that in the Stornorrfors tunnel, which is 52 ft. wide by 88 ft. high and is be-

lieved to be the largest water tunnel in the world, 2,270 tons of spoil are removed each day. But even the weight of spoil removed is not the ultimate criterion, for quite apart from the question of overbreak, every tunnelling contractor knows that the rate of advance is not in simple inverse arithmetical proportion to the tunnel cross section.

Natural conditions, too, are apt to have an overmastering influence on the rate of advance. The drilling characteristics of the rock, the need or otherwise for support, and the degree of liability to flooding, all influence the possibilities.

Indeed, the more the problem is considered, the more one becomes convinced of the virtual impossibility of devising any criterion by which tunnelling performance may be truly assessed and accurately compared.

Lest this should sound unduly pessimistic, let us hasten to add that there is a great deal to be gained by striving for record tunnelling performances. Naturally, the resulting prestige redounds to the advantage of the hydro-electric authority, the contractor, and the suppliers of the tunnelling equipment —and rightly so, because good tunnelling performance depends on choosing the right equipment for the particular job, the right personnel, and on first-class organisation throughout. The desire for achievement, too, inculcates that team spirit in which the best work is always done. Again, the time saved in tunnelling may reap handsome dividends in enabling the immense amount of capital locked up in a hydro-electric scheme to commence earning at an earlier date.

So let us continue to strive for newer and better records. Our only plea is that we should keep our feet firmly planted on the ground (or should it be, our gumboots firmly planted on the tunnel floor?) as to what exactly constitutes a tunnelling record, and just what it means when we have achieved it.

European Hydro Policy

AT a recent session of the United Nations Economic Commission the Committee dealing with electric power matters in Europe concluded that a growing emphasis would need to be placed on the development of thermal power coupled with the construction of pumped-storage hydro-electric plants to cover peak loads. It was also decided that the group of experts who are studying the hydro-electric resources of Europe should investigate the possibilities of developing multi-purpose projects on international rivers and the potentialities that exist in tidal energy.

Inga Congo Scheme

OFFICIAL approval has now been given by the Belgian Government to proceed with a gigantic hydro-electric development near Isangila in the Belgian Congo. This scheme has been under consideration since 1936, but it is only recently that detailed surveys have been undertaken and these are expected to be complete by 1959, when all will be in readiness to start the actual constructional work. The station will be situated at the downstream end of a loop of the Congo river about 100 miles from the coast and will have a capacity of 25,000 MW. At this point the river is estimated to have a flow of 700,000 cusecs and a head of 425 ft. will be made available. It is estimated that the total cost of the scheme will be in the neighbourhood of £1,128 million.

Finance will be arranged on an international basis and the power made available, estimated to be the cheapest in the world, will be used for new local industries, transmission to the Rhodesian Copper Belt and for enriching uranium.

Notes from Japan

A DAM for a multipurpose scheme has recently been completed at Ogochi, near Tokyo. It is of the concrete gravity type, 150 m. high, and the reservoir was due to be filled by the end of 1957. An arch dam 184 m. high is being built for Kurobe No. 4 project in the northern part of the Japanese Alps, and concrete dams are under construction for Fujiwaia, Akiba, and Ikawa, the last-named being of the hollow gravity type. At Takogura a 108 MW Francis set is being built, which will be the largest in Japan and, it is believed, will be the third largest in the world. Large schemes are also in progress in Tadami in north-eastern Japan, and Miboro in central Japan.

Northern Canada Power Commission

THE annual report of this Commission has now been issued and deals with the year ending March 31, 1957. Since its inception in 1949 the Commission has acquired and operates four plants having an aggregate of 12,700 h.p. Two of these—Snare River and Mayo River—are hydro-electric plants having a combined capacity of 11,350 h.p., while the other two are diesel plants with a total capacity of 1,350 h.p.

Growing demand required the installation of further generating capacity and to this end steps are being taken to erect a new 8,500 h.p. hydro-electric station on the Snare River some eight miles downstream of the present plant. It has also been decided to add a second 3,000 h.p. generating unit to the Mayo River station and to erect a new hydro plant at Whitehorse Rapids having an initial capacity of 15,000 h.p. in two units, with provision for a third unit to be added as need arises.

Transformers for Lewiston

FERRANTI LTD. have been awarded a contract valued at $3,925,250 by the Power Authority of the State of New York for 13 generator transformers. The contract covers six 180,000 kVA, 13·2/240 kV three-phase transformers and seven 180,000 kVA 13·2/120 kV three-phase transformers for the Lewiston power station of the Niagara power project situated on the plateau on the Niagara river about three miles below the Falls in the town of Lewiston. The transformers will be of the forced-oil, forced-water cooled, inert gas-filled type, and each will be directly coupled to a 167,000 kVA three-phase generator. Each transformer will weigh approximately 183 tons (240 kV) and 155 tons (120 kV). Delivery is to commence in the middle of 1959 and be completed by the end of 1960.

This contract, the twelfth awarded to Ferranti Ltd. by U.S. buyers in recent years, is believed to be the largest one for transformers so far placed in Britain by the U.S.A. It brings the total number of transfor-

mers supplied, or under construction by Ferranti Ltd. for the U.S.A., to 75 of a total capacity of nearly 5,000,000 kVA and a value of almost $10,000,000 (£3,571,428).

Pirttikoski Project, Finland

THIS underground power station, which is scheduled for completion in 1959, occupies a site upstream of a loop on the Kemi river and will utilise a head of 27 m. An annual output of 550 million kWh will be obtained, generated in two units having a capacity of 55 MW each. A mean annual flow of 283 cu. m. per sec. has been recorded over a period of 40 years and the catchment area is 27,700 sq. km. The impounding dam consists of an earth structure flanked by two concrete spillways built into the rock on either bank of the river.

A remarkable feature of this station is that it is situated in close proximity to the regulating dam, this arrangement entailing a tailrace tunnel 2,400 m. in length and having a cross-sectional area—second only to Stornorrfors in Sweden—of no less than 350 sq. m. This tunnel, which is being driven from both ends, will involve the removal of 890,000 cu. m. of rock which is being taken out in two stages, that is, by means of a top heading and bench. In addition to the tunnel, the open-air portion of the tailrace will be 530 m. in length and will entail the removal of 295,000 cu. m. of earth and rock.

Atiamuri Project, New Zealand

EXCAVATION work for the remainder of the concrete dam and the power house for the £7 million Atiamuri hydro-electric power project on the Waikato River, North Island, has now been completed. Mr. R. E. Hermans, the engineer in charge, said that, up to the present, progress had exceeded expectations. About 10,000 cu. yards of concrete is now being poured each month and the camp village is being extended to accommodate 30 more houses, making a total of 160. The labour force on the job has remained static around 600 since the beginning of the year.

At the Waipapa station, being built downstream from the completed Maraetai station, the first concrete in the diversion cut has been poured in the spillway block. Concreting of the 1,500 ft. cut is expected to take about a year and three new cranes are now being erected. Excavations for the power house are proceeding on the opposite side of the river and a sand plant for separating the fine pumice from the river sand is now operating.

Upstream from Atiamuri, concreting work on the 1,320 ft. diversion tunnel being built at the site of the Ohakuri dam is two-thirds completed. The Ministry of Works, operating on the opposite side of the river, is excavating for the power house, the forebay for the intake dam, the tailrace and the control block.

Mapping Europe's Hydro-electrical Potential

AS part of the activities of the Committee on Electric Power of the Economic Commission for Europe, a Group of Experts for the Study of Hydro-electric Resources in Europe met at Geneva from October 7 to 9, 1957, and adopted a provisional map showing in standardised form the distribution of hydro potential, as represented by known mean stream flows and differences of elevation, over a large area of Continental Europe. This map, the first of its kind, will be progressively extended as further data are received from the countries concerned. Since the scale of the map is 1 : 2,500,000, insets at a somewhat larger scale are to be included for particular areas where this is necessitated by the density of the isopleths.

The Group of Experts also continued its study of methods of determining net hydro-electric potential and of comparing gross and net potentials. The fact that a high percentage of the total hydro resources of such countries as the Federal Republic of Germany, Italy and Switzerland, were already developed was recognised as giving added interest for less-developed countries to the correlation between theoretical and developed potentials in such cases.

The Group of Experts stressed the importance for Europe's energy supply of obtaining increased knowledge of the regional patterns of irregularity in stream flow. It also decided to request the Geophysical Institute of Trieste to extend to the whole of Europe the study of fluctuations already carried out in Italy, and to ask the ECE Secretariat to continue its enquiry into the stream-flow data already available with a view to the preparation of a first map of Europe showing the main hydrographic regions.

Owen Falls—Kenya Transmission Line

THE UGANDA ELECTRICITY BOARD have now energised the Kenya Power Company's main 132 kV transmission line. For the past 12 to 18 months Balfour Beatty & Co. Ltd. have been engaged on the construction of this line on behalf of the Kenya Power Company Limited and the Uganda Electricity Board. The line, which extends for a distance of some 250 miles from a switching station at Tororo in Uganda to Nairobi, will transmit the power from the Owen Falls power station.

Actual construction of the line has not been without its difficulties, but the contractors have been able to keep ahead of schedule. The technical problems associated with the transmission of the supply have for some time past been engaging the attention of the Board and the Power Company in consultation with experts in the United Kingdom, but these have now been resolved.

When the Uganda Electricity Board first started operating in 1948, about 500,000 kWh per month were generated; now the figure is over 14 million kWh, representing a 28-fold increase in 10 years. All classes of consumers, it is stated, are demanding more electricity.

Harrsele Station

IN connection with the note in our November issue about the commissioning of Harrsele station, we have now been informed that although the Pengfors, Harrsele and Bjurfors Nedre stations have been, or are being, equipped with KMW/ASEA generating sets, the orders for the generating sets for the projected Bjurfors Övre station have not yet been placed. It will be remembered, of course, that the Stornorrfors station, referred to in the last paragraph of our note, is to be equipped with three 178,000 h.p. 125 r.p.m. Nydqvist & Holm turbines.

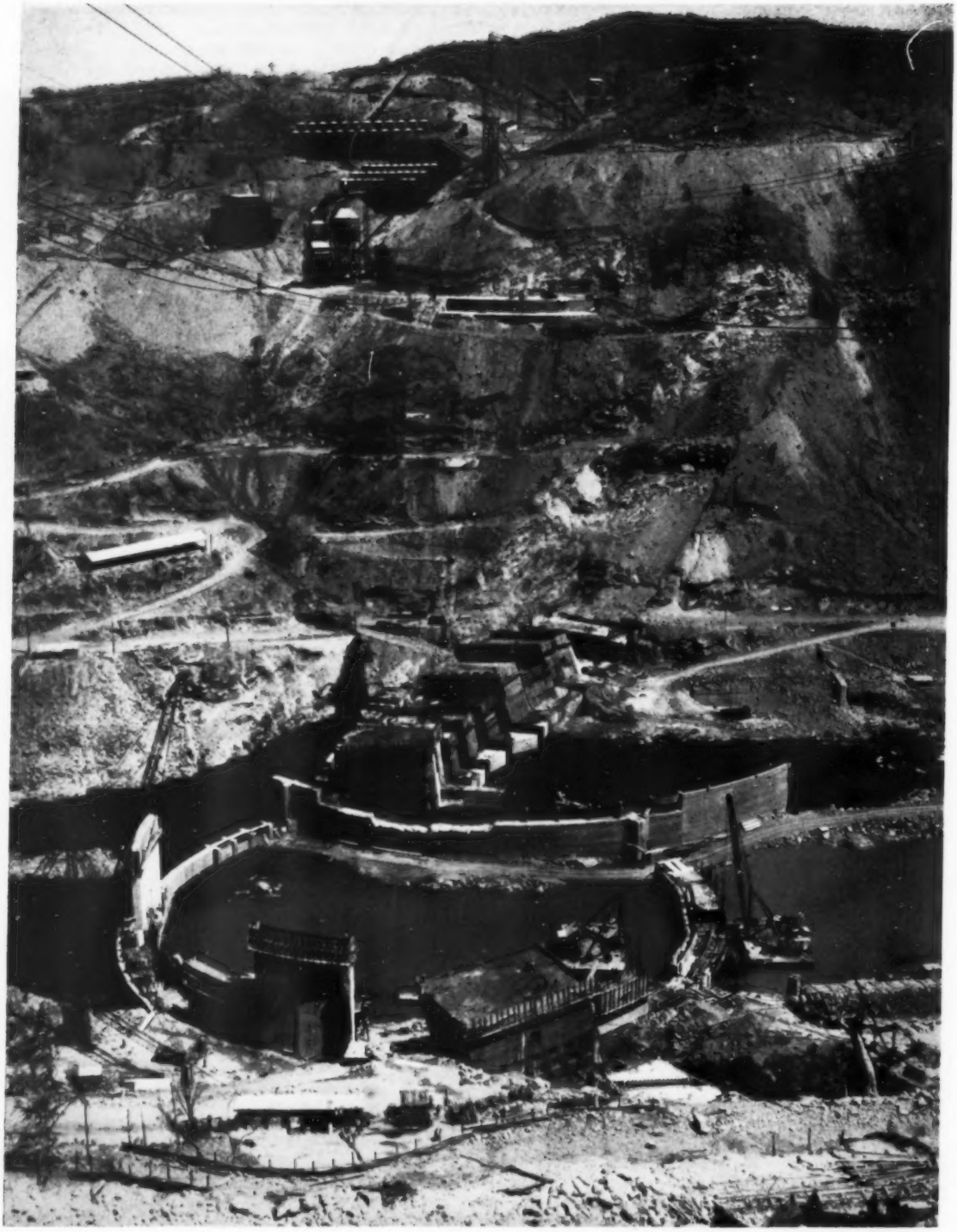


Fig. 1. Recent view looking north-west showing walls of first cofferdam partly demolished and second cofferdam in course of construction

The Kariba Hydro-Electric Scheme

By SIR ALEXANDER GIBB AND PARTNERS

PART ONE

THE Kariba gorge is on the Zambezi where the river forms the border between Northern and Southern Rhodesia, about 300 miles downstream from the Victoria Falls. The dam now under construction here will create one of the greatest man-made lakes in the world. Though the natural flow of the river is subject to very wide seasonal variations, the huge capacity of the reservoir will make a large volume of water available throughout the year for power generation. The ultimate installed capacity will be at least 1,200 MW with a mean head of 310 ft., and annual output is estimated at 8,180 million kWh.

Apart from size, one of its special points of interest is the method of construction, which has been devised

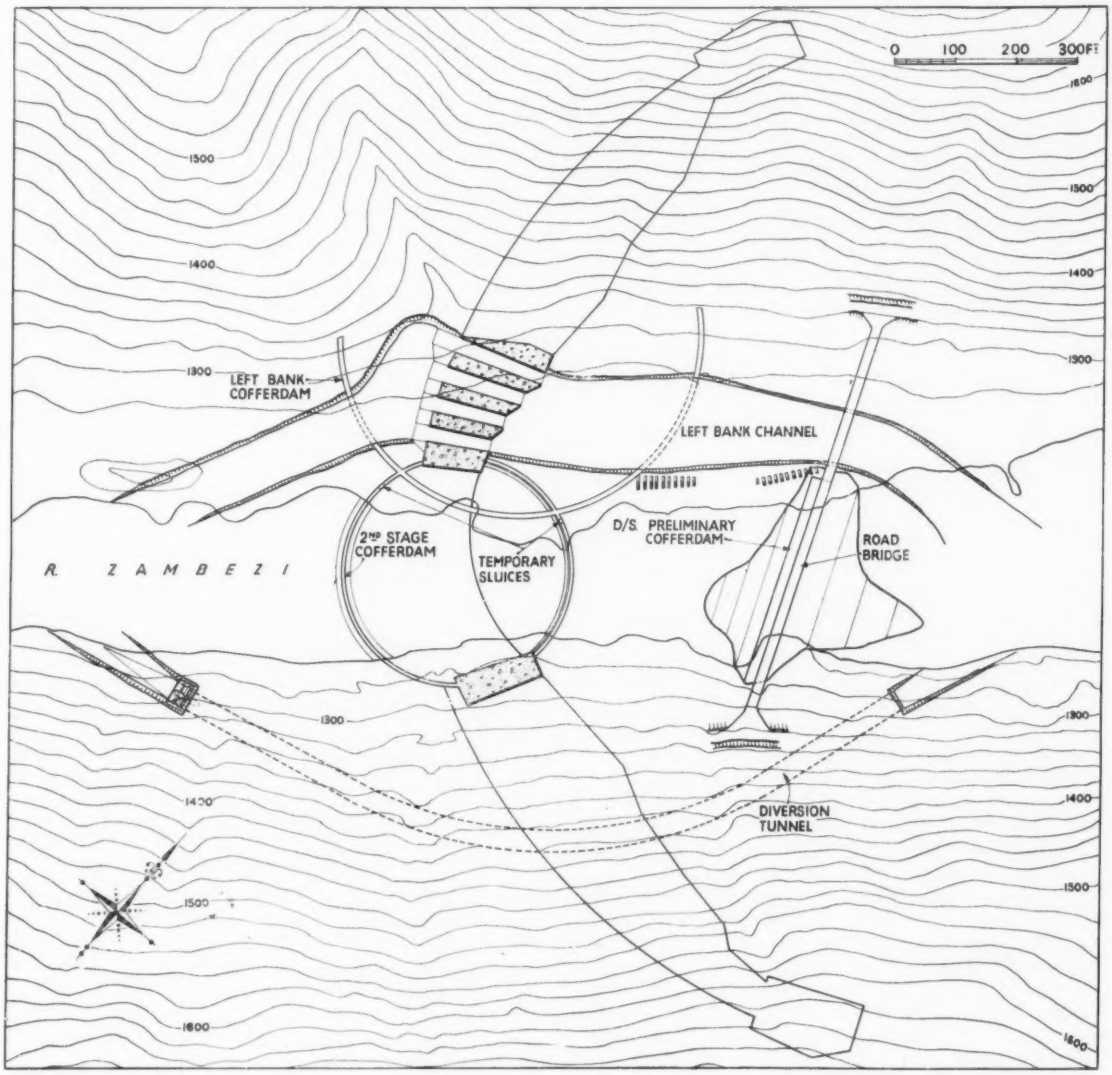


Fig. 2. General plan of operations

Fig. 3. Aerial view, looking upstream, with downstream end of diversion tunnel and road bridge in foreground

to meet particular site conditions and to provide for the early production of a limited supply of power.

Demand

Industrial development in the territories comprising the Federation of Rhodesia and Nyasaland has been extraordinarily rapid in recent years. The mainstay, of course, is copper; but there are many other minerals and a growing number of secondary industries. The chief industries are contained within the triangle Kitwe-Bulawayo-Salisbury; and it is fortunate that Kariba lies very near the centre of this area.

Expansion, which is likely to continue at least at the present rate (the Copperbelt, for instance, is engaged in development to achieve big increases in output by 1960), is dependent on ample supplies of power. Electricity consumption in Northern and Southern Rhodesia, which was 1,800 million kWh in 1954, is expected to reach 3,200 million by 1960 and 6,500 million ten years later.

Hitherto Northern and Southern Rhodesia have drawn their supplies of electricity mainly from thermal stations. Any addition to these would involve further strain on the already hard-worked railways, for the only coalfield actually developed in the Federation is far from the industrial centres. Nuclear generation was not sufficiently advanced to be considered at this stage. The general advantages of hydro-electric power, especially for peak-load purposes, were enhanced by the presence of a great river.

Two sites particularly attracted attention: one on the Kafue river, in Northern Rhodesia; and the other on the Zambezi at the Kariba gorge. A Commission appointed by the Central African Council reported in 1951 in favour of Kariba as more suitable for immediate development; this opinion was confirmed by an independent mission of French experts and subsequently by M. André Coyne, the well-known French consulting engineer. On March 1, 1955, the then Prime Minister of the Federation, Sir Godfrey Huggins (now Lord Malvern), announced the decision to proceed with Kariba. He said that action must be taken quickly so that work could commence on the site as soon as weather conditions permitted; i.e. about the end of June.

On June 7, the Federal Hydro-Electric Board (later the Federal Power Board) made known the appointment, as joint civil engineering consultants for the first stage of the Kariba scheme, of Sir Alexander Gibb & Partners, Messrs. André Coyne and Jean Bellier, and the Société Générale d'Exploitations Industrielles. Shortly afterwards the appointment of Messrs. Merz and McLellan as mechanical and electrical consulting engineers was announced. Everything possible was done to facilitate the early placing of contracts and a start was actually made with preparatory work on the site in mid-July 1955. Meanwhile, the financial aspects of the scheme were receiving attention: negotiations for loans were in progress with the World Bank, the Colonial Development Corporation, the Commonwealth Development Finance Company, the Northern Rhodesia copper companies, the British South Africa Company and the commercial banks.

It will be convenient first to outline the complete

scheme as designed before describing the construction programme.

Hydrology

The flow of the Zambesi at Kariba varies on average between 16,000 cusecs in the dry season and 200,000 in the wet season, which falls between December and June, with maxima of the order of 300,000 cusecs over short periods. This large seasonal variation, combined with the need to make full use of all available water for power production at a fairly even rate throughout the year, made very large storage capacity essential. Fortunately the topography of the land upstream from Kariba constitutes an enormous natural basin (it was in fact a lake at one time in the distant past) and the gorge is sufficiently narrow and steep-sided to be spanned by a dam of very moderate length.

The Dam

The chosen site is 2½ miles downstream from the entrance to the gorge, where drilling proved sound rock at river-bed level. A few extra feet of head might have been gained by building at the downstream end of the gorge, but thick deposits of gravel or sand would have made construction more difficult and—most important—might have delayed completion.

The dam will be a pure arch structure in mass concrete standing about 420 ft. high from its foundations. Its maximum thickness will be about 80 ft., compared with about 280 ft. required for a gravity dam, thus allowing substantial economies in labour and materials. The length of the roadway above the crest will be some 1,900 ft. To provide for spillage—which may be substantial during the period of load build-up, though it will be kept to a minimum once the scheme is working to full capacity—there will be four floodgates in the dam wall each 31 ft. wide and 30 ft. high with sill level at about 92 ft. below normal top water level. These together, are designed to pass up to 230,000 cusecs, which is adequate to route through the reservoir the size of flood which might occur once in 10,000 years.

The discharge from the floodgates will fall freely some 200 ft. in the form of a jet landing 300 ft. horizontally away from the base of the dam—an impressive sight for the tourists whom it is hoped the scheme will attract in large numbers, but after all the plant is installed the dam may seldom spill. The foundations of the dam will be protected by a concrete apron.

Structural tests of the dam design have been carried out in Paris by SEIL, assisted by the consultants, by means of a scale model constructed of plaster of paris and diatomite. The tests were of great assistance in reducing the time necessary for preparing the final designs.

Hydraulic model experiments are in progress both at the Laboratory of SOGREAH at Grenoble where various methods of diverting the river during construction have been tested as well as designs for the flood gates; and at the British Hydromechanics Research Association laboratory at Harlow where the intakes for the power station have been studied.

Fig. 4. Blondin platform on south bank, looking downstream

The Reservoir

The dam will raise the water about 350 ft. above the dry-season level of the river, and will create a reservoir 175 miles long extending upstream to the lower end of the gorge below the Victoria Falls. The Kariba lake will have a surface area of 2,000 sq. miles and a capacity of some 130 million acre-feet—more than four times the amount impounded by the Hoover dam which is at present the largest artificial reservoir in the world. The active storage, that is to say the storage available for power and for flood control, will be 20 million acre-feet. The dam will provide a high degree of regulation in spite of the natural seasonal variations: 84½% of the total river flow will be available for power generation, allowing for 9½% loss by evaporation and only 6% passing through the floodgates. Thus, it will be pos-

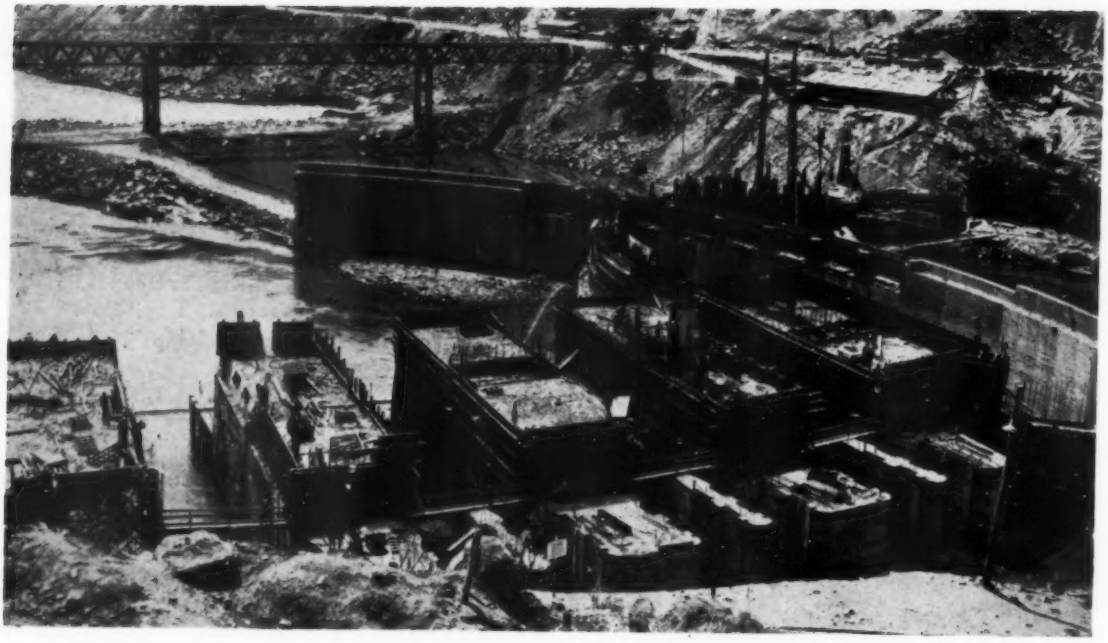


Fig. 5. View inside first stage cofferdam showing temporary openings in main dam

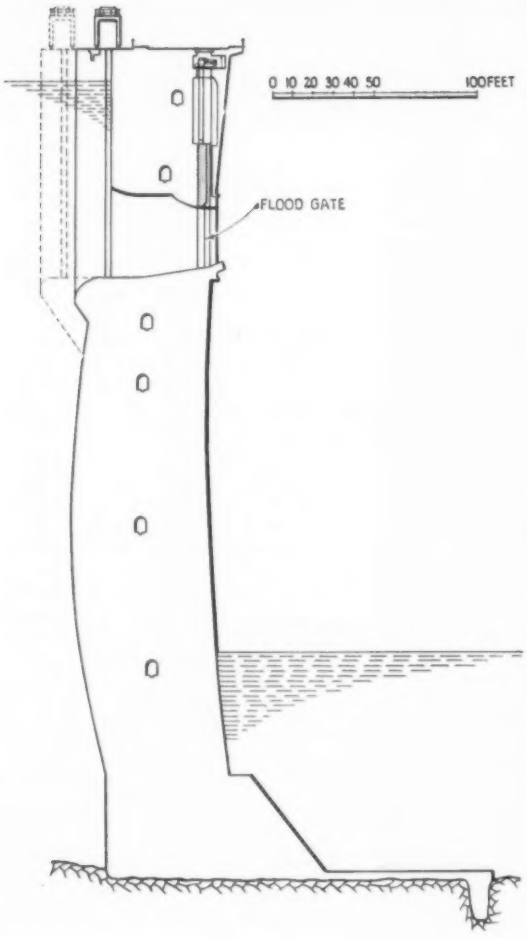


Fig. 6 (left). Diagrammatic section through Kariba dam showing flood gates

sible to secure a continuous firm flow through the power stations of 40,000 cusecs, which is equivalent to an annual energy output of 8,180 million kWh per year.

The scheme will inevitably influence the regime of the river downstream and this will affect Mozambique. It has been agreed with the Portuguese Government that a specified minimum flow will be maintained throughout construction. After completion the controlled discharge will be a great advantage to Mozambique; navigation will no longer be subject to the almost complete stoppages that have occurred in past seasons of exceptionally low flow.

One of the many incidental problems demanding attention is concerned with the papyrus reeds that grow on the exposed river banks in the dry season. These are uprooted when the river rises and form floating islands sufficiently substantial to support the cattle which are sometimes found marooned upon them. Measures have been under consideration to ensure that these floating masses will not accumulate over the power station intakes.

The flooding of very large areas will involve the displacement of about 51,000 Africans, mostly of the primitive Batonka tribe, who live by simple cultivation of soil which is generally poor. Owing to the rapid river current, they have not become dependent to any extent on fishing. Early one morning in August 1955, a small party under the Southern Rhodesian Minister of Native Affairs set out to travel through hundreds of miles of country little known to white men. Their task—not an easy one, it must be admitted—was to explain to the local chiefs why they and

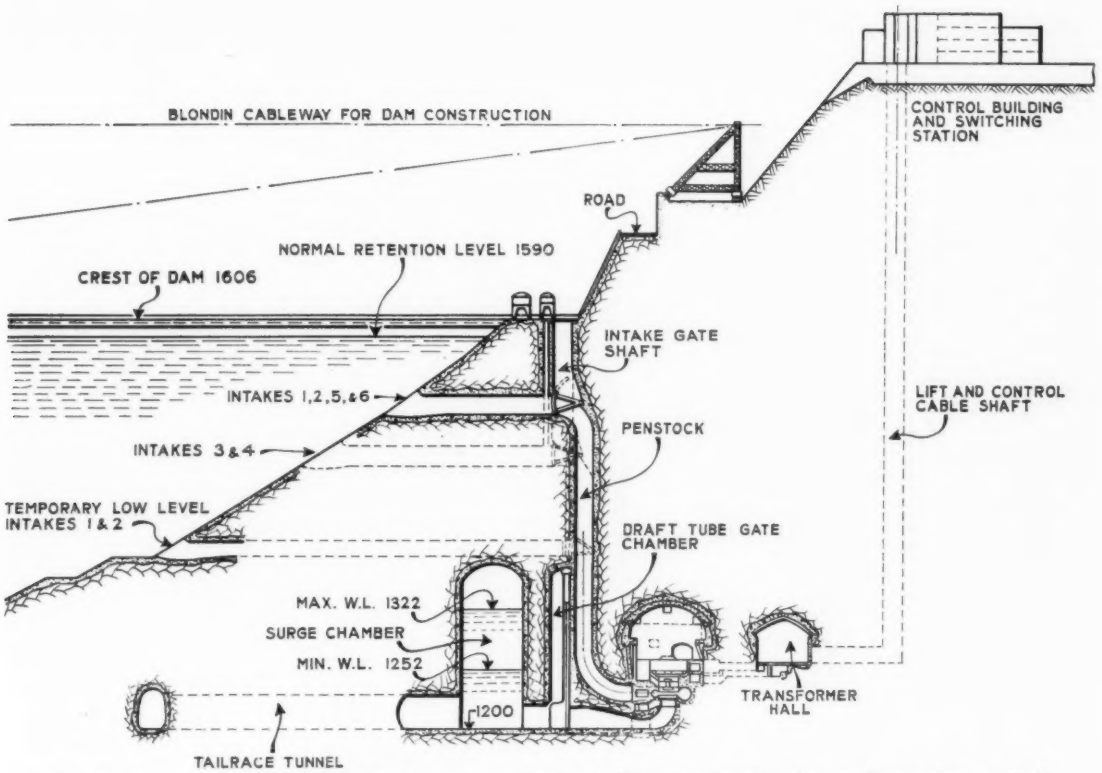


Fig. 7. Cross section showing intakes, power house, tailrace surge chamber and transformer hall

their people were soon to be asked to leave the territories they have occupied in contented isolation for generations and move to new areas that have been provided for them.

The Power Stations

The first stage of development comprises the construction and equipment, as well as the dam, of an underground power station in the right bank. This will be 460 ft. in length, 75 ft. wide and 125 ft. high. It will be placed immediately upstream of the dam abutment, and this unusual arrangement has involved some complications in design; the intakes and draft tubes, for instance, will be on the same side of the machines, and the underground works will be subjected to water pressures due to the reservoir above. Access to the power station for heavy machinery, etc. will be by a sloping tunnel. Two electric lifts will communicate with the control room and offices at ground level nearly 600 ft. vertically above.

Ultimately there will be six intakes, one for each machine,

Fig. 8. Concreting soffit to No. 2 temporary intake

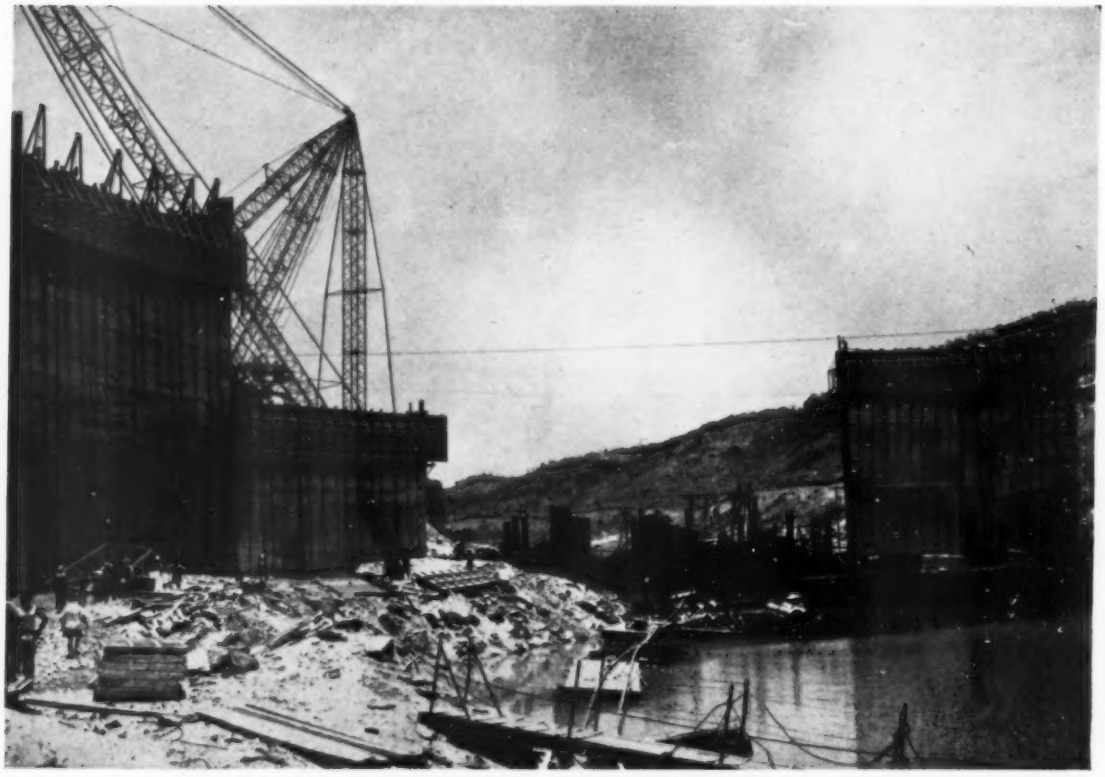


Fig. 9. Placing concrete in second-stage cofferdam, looking upstream

set in the sloping face of the gorge, but in order to operate two machines at an early date before the water is raised to its final level, two temporary low-level intakes will also be constructed: these will later be sealed off. The permanent intakes will lead by short horizontal tunnels into vertical shafts, of which the upper part will be lined with concrete and the lower, of reduced diameter, with steel. Radial intake gates are set in the junction between the horizontal intake tunnels and the vertical shafts. The gates are operated from a platform 16 ft. above normal top water level.

Discharge will be by three tailrace pressure tunnels provided with surge chambers. The tunnels are of horseshoe section having an equivalent diameter of 34 ft., and will be unlined for most of their length.

Plant

The six water turbines will be of the vertical-shaft Francis type, designed for an output of 140,000 b.h.p. each at a net head of 282 ft. At present a contract for five machines has been awarded to Boving & Co. Ltd.

The generators, which will be direct coupled to the turbines, will be designed for a continuous maximum output of 100,000 kW at 0·9 power factor, three phase 50 cycles. They will be of the synchronous closed-air-circuit, water-cooled type. Owing to their size it will not be possible to erect and test them fully in the manufacturers' works. They will be split into components for transport, and the rotors, which will weigh about 380 tons each, will be completely assembled on site. The initial five generators are being manufactured by Metropolitan Vickers Electrical Export Co. Ltd.

Voltage will be stepped up from 18 kV at the generators to 330 kV by single-phase transformers which will occupy a separate underground gallery alongside the machine hall. Output will then be carried by oil-filled cable to an open-air switching station for transmission. The transformers are being supplied by Ferranti Limited.

Second Stage

The second stage envisages a further underground power station to be built in the left bank of the river, giving an ultimate total installed capacity of at least 1,200 MW.

Transmission

Transmission lines will run from Kariba northwards to Kitwe (for the Copperbelt) and southwards to Umniati and Norton. Another line connecting Umniati and Norton will continue north-east to Salisbury and south-west to Bulawayo. The total length of these lines will be 935 miles. The conductors will be of steel-cored aluminium. (Although the Federation is the second largest copper producer in the world, the use of copper conductors was ruled out on the ground of price.) There will be two conductors per phase and two earth wires.

Materials and Labour

The construction of the dam, which is scheduled for completion by August 1960, will involve the plac-

Fig. 10. Off-loading point for cement transported in bulk containers

ing of 1,400,000 cu. yards of concrete in 3½ years, reaching a peak of about 2,500 cu. yards per day. The right-bank power station with its intakes, penstocks, tailraces and access tunnel will require the excavation of well over one million cu. yards of rock and the pouring of 160,000 cu. yards of concrete. The largest single item on the list of materials required is cement, which amounts to about 400,000 tons; this can be supplied by factories within the Federation. Coarse aggregates will be quarried near the dam, and the sand will mainly come from river deposits. Consumption of petroleum fuels (of which the chief is diesel oil) will exceed 10 million gallons. Steel requirements are comparatively modest and are not expected to present a considerable problem. Contractors' plant, estimated at 9,000 tons by 1957, will be largely imported.

In all, some 600,000 tons of goods will move to Kariba by the end of 1960, with a peak of 184,000 tons in 1959. Less than one-third of this will travel by rail, and in any case it will not constitute a large additional burden on the railways, which have carried as much as 10 million tons in a year. Road communications with Kariba itself, on the other hand, were quite inadequate and new roads from north and south had to be provided. Together these amounted to nearly 100 miles of new construction.

The intake of African labour is not expected to create a shortage elsewhere. The maximum number required will be under 7,000 as against a total of well over a million in paid employment in the Federation. The Africans working on the scheme will benefit not only by steady employment and high wages but also by the oportunity to acquire new skills.

The European population at Kariba during construction may reach 1,500 including families; but even if all these were immigrants the figure is less than one-tenth of the annual intake for which the Federation has planned. On the other hand the accommodation of some 10,000 people in a remote place, hitherto scarcely inhabitated, is a considerable undertaking in itself.

In a subsequent article it is proposed to describe the construction programme and other interesting aspects of the scheme.

(To be continued)

Fish Preservation in Scotland

The North of Scotland Hydro-Electric Board have completed their harvest of fertilised salmon ova for the year and have something like two million of them safely stored in plastic trays, which are sunk in troughs and continuously washed in cold water, in their Invergarry hatchery. Most of these ova will be released in the spring but a proportion will be kept as yearlings and others will not be released until a further period of storage converts them into smolt. The two-year olds will be tagged before release. This hatchery is part of the Board's programme of fish preservation and co-operation with angling activities. In building the Garry and Quoich dams the loch levels in each case were raised substantially and spawning areas flooded. A fish lift was incorporated in the Garry dam and at Quoich, on the Upper Garry, the Board laid down an electrode barrier which checks the fish in a holding pool on Loch Poulary. A fish heck was laid half a mile upstream with three bird-cage fish traps and there the salmon are stripped, the eggs and milt then being artificially handled and the fish returned to the river.

The Upper Garry is not regarded as a very important angling stretch as the 50-mile run and a climb of 350 ft. discourage the fish. The interesting point will therefore be the extent to which the new conditions will influence the fish population of the area. It is anticipated that heavier runs will occur up the Ness during the next two or three years and neighbouring rivers may also benefit.

Effect of Pore Pressure on Stress Distribution in some Porous Elastic Solids

By O. C. ZIENKIEWICZ, Ph.D., A.M.I.C.E., and JAMES PARK, M.Sc.

This article presents some solutions of stress distribution due to pore pressure in dams and in thick cylinders as derived by the authors. Some of the results here have not been published previously; others form the basis of a paper published recently by the Am. Soc. C.E.(7)

WATER-PRESSURE present in the pores of concrete or rock is a well-known factor affecting the distribution of stress, and as such it is always considered in the design of hydraulic structures. Uplift stresses in dams are just one of the many manifestations of this wide problem. It is proposed in this article to consider an analytical method by which such stresses can be evaluated—at least in two-dimensional problems—and to consider some implications of the results obtained. As in all analytical approaches to a stress problem, certain idealising assumptions have to be made regarding the basic material of the structure. Although some of these—such as an assumption of elasticity and homogeneous nature—are only coarse approximations to the performance of the most usual material concerned, namely concrete, the results are nevertheless the nearest approximation to the solution that can be expected in the circumstances.

NOTATION

p	pore pressure
∇^2	$\left(\frac{\partial^2}{\partial x^2}+\frac{\partial^2}{\partial y^2}\right)=\left(\frac{\partial^2}{\partial r^2}+\frac{1}{r}\frac{\partial}{\partial r}+\frac{1}{r^2}\frac{\partial^2}{\partial \theta^2}\right)$
∇^4	$\nabla^2 . \nabla^2$
x, y	Cartesian co-ordinates
r, θ	polar co-ordinates
η	area factor
w	unit weight of water
w_s	unit saturated weight of porous material
γ	w_s/w
X, Y, R	body forces in the directions of x, y and r, respectively
$\sigma_x, \sigma_y, \tau_{xy}$	Cartesian stress components in two-dimensional plane strain system
$\sigma_r, \sigma_\theta, \tau_{r\theta}$	polar stress components in two-dimensional plane strain system
ϵ_r, ϵ_t	components of radial and tangential strain in polar system
ν	Poisson's ratio of porous material
k	$\eta\frac{(1-2\nu)}{(1-\nu)}$
ϕ	Airy's stress function
α	apex angle of triangular dam
β	drainage angle in triangular dam
ψ	function of θ governing pressure distribution in a dam
n	pressure reduction factor
F	a stress function
H	unit step function

Pore pressure as a body force

It is common practice to assume that the horizontal thrust of water in a reservoir is applied entirely at the upstream face of a dam, or that water pressure is exerted entirely at the inner surface of a concrete pipe. However, it has frequently been demonstrated in laboratory tests and by observations in large concrete structures that the material is not entirely impermeable and that appreciable internal pore pressures are built up indicating a slow seepage of water through the concrete. Thus the pressure of fluid at the surface where it enters the body is gradually reduced by friction within the pores until it reaches the value of the pressure at the outlet surface, which is either the atmosphere or some internal drainage boundary. The force applied to the structure is therefore transmitted to it along the lines of seepage as a body force, and not by boundary loading. Put in analytical terms this means that an elementary volume of the porous body (as shown in Fig. 1) experiences forces in the x and y directions, respectively, of:

$$X=-\eta\frac{\partial p}{\partial x}$$

$$Y=-\eta\frac{\partial p}{\partial y}+w_s$$

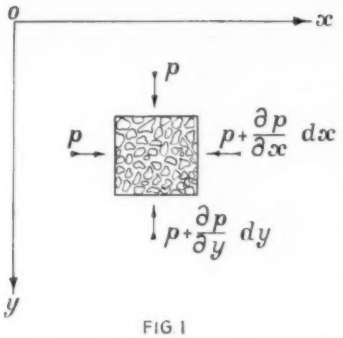


FIG. 1

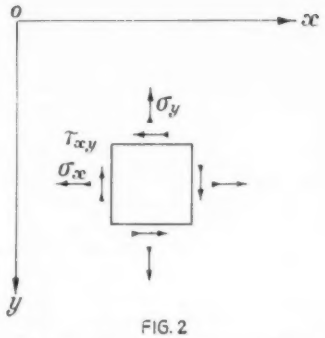


FIG. 2

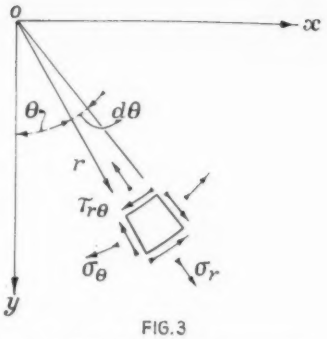


FIG. 3

where y is the direction of gravity and η is the proportion of the area of the element on which pore pressure is effective.

This quantity η, generally known as the area factor and sometimes as the boundary or superficial porosity, is one the magnitude of which has been in dispute since pore pressure and uplift forces were first discussed. It was initially believed that η was equal to the porosity of a plane section through the material, which, in the case of concrete, would be 12 to 15%; but subsequent laboratory tests designed to measure the magnitude of the area factor indicated that it had a much higher value, probably in the region of 90 to 100%.[1,2] These tests generally involved the measurement of forces on a specimen at the moment of rupture and there was always some doubt whether the results were applicable to cases where rupture was not involved. Theories were advanced to explain the high values, generally on the basis that the pore-pressure effect was similar to the phenomenon of buoyancy. The most recent experimental work[3] indicates that the area factor has a somewhat lower value, but although this is the most realistic work done so far, involving measurements under steady conditions of seepage without rupture, the evidence is by no means conclusive. It is doubtful if the solution will be finally obtained until a thorough understanding is reached of the complex physical structure of cement and concrete. In the meantime, and for the purpose of this article, there appears no justification for using a value less than the safe maximum of 100%.

General Theory for Two-dimensional Stress Analysis

Defining the stress components as in Fig. 2, the conditions for equilibrium of an elementary volume (including the body forces) are:

$$\frac{\partial \sigma_x}{\partial x}+\frac{\partial \tau_{xy}}{\partial y}-\eta\frac{\partial p}{\partial x}=0$$

$$\frac{\partial \sigma_y}{\partial y}+\frac{\partial \tau_{xy}}{\partial x}-\eta\frac{\partial p}{\partial y}+w_s=0$$

It is necessary for the stress components to satisfy the condition for compatibility of strain, which, in a two-dimensional system of plane strain, is:[4]

$$\nabla^2(\sigma_x+\sigma_y)=-\frac{1}{(1-\nu)}\left(\frac{\partial X}{\partial x}-\frac{\partial Y}{\partial y}\right)=\frac{\eta}{(1-\nu)}\nabla^2 p \quad ...(1)$$

Introducing a stress function ϕ, the equations of equilibrium are automatically satisfied if the stress components are defined in the following manner:

$$\sigma_x=\frac{\partial^2\phi}{\partial y^2}-w_s y+\eta p$$

$$\sigma_y=\frac{\partial^2\phi}{\partial x^2}-w_s y+\eta p \quad ...(2)$$

$$\tau_{yx}=-\frac{\partial^2\phi}{\partial x\,\partial y}$$

Substituting expressions (2) into (1) leads to the relation:

$$\nabla^2(\nabla^2\phi+kp)=0 \quad ...(3)$$

where $k=\eta(1-2\nu)/(1-\nu)$. A function ϕ satisfying the above equation and the boundary conditions of a problem determines the solution.

It is a straightforward matter, by expressing (1) and (2) in polar instead of Cartesian co-ordinates (and defining the stress components as shown in Fig. 3), to show that equation (3) is valid for either system.

Brahtz' theory and its application

The first and most widely accepted general method of treatment of stresses in porous bodies was given by Brahtz[5] whose simple hypothesis has been widely used and misused. His conclusions can be summarised as follows:—

To find the two-dimensional stress distribution in an elastic body caused by the action of internal pore pressures, it is permissible to find first the stress distribution in the body assumed impermeable with appropriate pressure applied to its boundaries, and then add to the values of stress thus obtained the pore pressure multiplied by the area factor.

It is a necessary condition of Brahtz' theory that seepage should obey Darcy's law throughout the region considered, which is equivalent to the pressure function p obeying the Laplace equation:

$$\nabla^2 p=0$$

Proof of the above hypothesis can be deduced directly from equation (3). For a non-porous body, the term $\nabla^2 p$ disappears and the governing relation becomes $\nabla^4\phi_1=0$. For a porous body, in regions of continuous flow, $\nabla^2 p=0$ and so the governing relation is again $\nabla^4\phi_2=0$. The general solution to this latter relation contains several arbitrary constants which must be determined from the boundary conditions. Considering a boundary $x=0$ where the applied pressure is p_0, it is seen that for a non-porous body the boundary condition is:

$$\sigma_x=-p_0$$

[1,2] Figures in brackets refer to the bibliography at the end of the article.

and for a porous body it becomes:

$$\sigma_x = -p_0(1-\eta)$$

Using these conditions in the first of equations (2), for a non-porous body

$$\sigma_x = \frac{\partial^2\phi_1}{\partial y^2} - w_s y = -p_0$$

or $$\frac{\partial^2\phi_1}{\partial y^2} = w_s y - p_0$$

For a porous body:

$$\sigma_x = \frac{\partial^2\phi_2}{\partial y^2} - w_s y + \eta p_0 = -p_0(1-\eta)$$

or $$\frac{\partial^2\phi_2}{\partial y^2} = w_s y - p_0$$

Thus $\phi_1 = \phi_2$ because the boundary conditions will give identical values of the arbitrary constants in the general solution of ϕ for porous and non-porous bodies. Brahtz' conclusion then follows from equation (2) from which it is now seen that because ϕ is identical for the porous and non-porous cases, the only difference in the normal stress components is the term ηp. The simple hypothesis outlined above has been shown by Serafim[3] not to be valid for cases of three-dimensional stress distribution.

As an example of the application of Brahtz' theory, consider a horizontal section of a triangular dam profile with co-ordinates as shown in Fig. 4. The uplift pressure must be distributed as shown in Fig. 4a to satisfy the Laplace relation. The distribution of vertical stress in the absence of pore pressure approximates elastically to the type shown in Fig 4*b* in portions of the dam not affected by foundation contact. Thus, applying Brahtz' result, the net vertical stress is obtained by subtracting the pressure p in Fig. 4*a*, multiplied by the area factor η, from the stresses in Fig. 4*b* to give the distribution of stress shown in Fig. 4*c*.

However, if a system of internal drainage within the dam were to create a drainage plane at some angle β to the upstream face (as shown in Fig. 5), the distribution of pore pressure would be that shown in Fig. 5*a*. The pressure distribution no longer satisfies the condition $\nabla^2 p = 0$ on the line of drains, and the question arises whether Brahtz' result can be extended to this case. To do so would give a non-linear distribution of stress as shown in Fig. 5*c*, from which one could conclude that the effect of internal drainage had in no way relieved the stress condition at the upstream face of the dam. The analysis in the next section shows that Brahtz' theory must not in fact be extended beyond the strict limits of its applicability recognised by its author, and treats the above case from basic first principles.

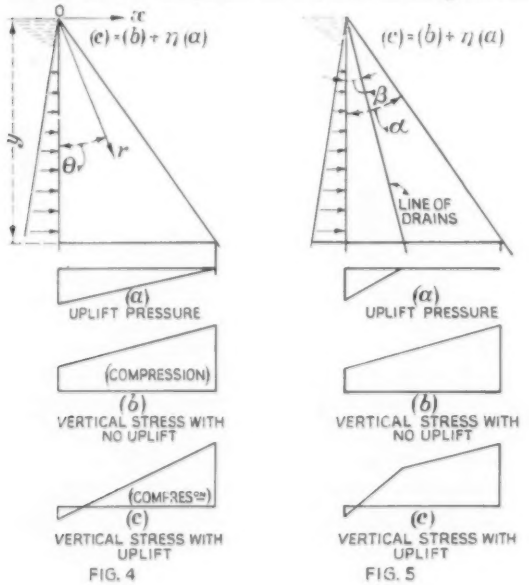


FIG. 4

FIG. 5

A less obvious limitation to the application of Brahtz' theory is to be found in the analysis of multiply-connected bodies (i.e., bodies with closed internal boundaries) such as, for example, the cylindrical sections to be considered later. It is well known from the basic concepts of the theory of elasticity that a unique solution of equation (3) cannot be determined in a multiply-connected body without a consideration of displacements. It is found therefore that in general the solution will contain the elastic constants of the material, and therefore the arguments which lead to the establishment of Brahtz' theory are no longer valid. A proof of the above limitation can be obtained easily by consideration of the stresses in a cylindrical section caused by an axially symmetrical pore-pressure distribution which satisfies the Laplace equation.

The general solution of equation (3) for this case is:

$$\phi = A \log r + Br^2 \log r + Cr^2 + D \qquad \ldots(4)$$

Rewriting the stress components (2) in polar co-ordinates for this case of symmetry results in:

$$\sigma_r = \frac{1}{r}\frac{d\phi}{dr} + \eta p$$

$$\sigma_\theta = \frac{d^2\phi}{dr^2} + \eta p$$

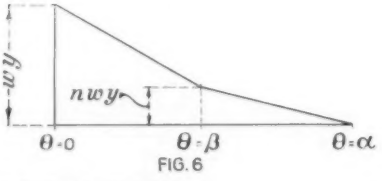


FIG. 6

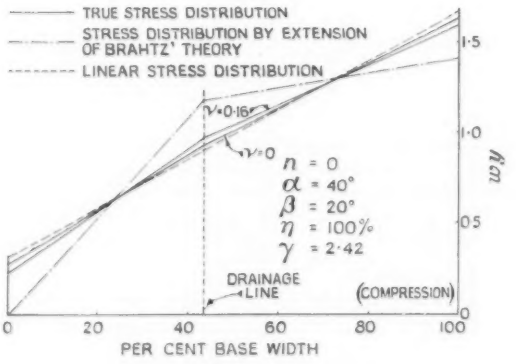


FIG. 7

For a unique solution it is necessary to determine the arbitrary constants A, B and C in equation (4), but there are only two boundary conditions, namely the conditions of normal stress at the inner and outer boundaries. In order to find a third condition, it is necessary to consider displacements, imposing the condition that they must be single-valued functions only. In this way a factor is introduced which is foreign to Brahtz' proof and which leads to different values of the arbitrary constants for porous and non-porous sections. This is demonstrated in the solution to case 1 of radial percolation of water in a cylinder given later. That the foregoing limitation of Brahtz' theory is not obvious is apparent from a recent paper by Gherardelli[6] where the very problem demonstrated above is discussed and erroneous results obtained.

Distribution of Stress in a Triangular Dam Profile†

For the triangular dam profile shown in Fig. 5, it follows from consideration of symmetry that stresses on similar (parallel) sections must follow similar patterns; that is to say, the stress at any point can be defined by the product of two independent functions of r and θ in the form:

$$r\,F(\theta)$$

By considering the stress relationships (2), the stress function ϕ can be shown to have the general form:

$$\Phi = r^3 f(\theta) \qquad \ldots(5)$$

Also, since pore pressure increases linearly with depth, the pressure distribution can be defined quite generally in the form:

$$p = r\,\psi(\theta) \qquad \ldots(6)$$

Both f and ψ are functions of θ only.

Substituting (5) and (6) into the governing relation (3) gives the following ordinary linear differential equation which can be solved for any assumed pressure distribution defined by the function ψ.

$$\frac{d^4f}{d\theta^4}+10\frac{d^2f}{d\theta^2}+9f=-k\left(\psi+\frac{d^2\psi}{d\theta^2}\right) \qquad \ldots(7)$$

The general solution of (7) has the form:

$$f = A\cos\theta + B\sin\theta + C\cos 3\theta + D\sin 3\theta + \text{Particular Integral}$$

Consideration of the boundary conditions leads to the evaluation of the arbitrary constants A, B, C and D, and the particular integral is derived from the assembled form for the pressure distribution. For various forms of the function ψ different mathematical difficulties have to be overcome, but in general a formal solution is always possible.

A case of considerable interest and generality is presented in Fig. 6. Here, a line of drains situated on a plane passing through the apex of the dam reduces pore pressure to an nth fraction of the upstream value. With the exception of this line the material is assumed homogeneous and the seepage obeys Darcy's law. This pressure distribution results in the following form of the function ψ:

$$\psi = w\left(\cos\theta + \frac{(n-1)\sin\theta\cos\beta}{\sin\beta}\right) \quad \text{for } 0<\theta<\beta$$

$$\psi = \frac{wn\cos\beta\sin(\alpha-\theta)}{\sin(\alpha-\beta)} \quad \text{for } \beta<\theta<\alpha$$

The resulting distribution of stress is found to be given by two straight lines of different gradients in the regions on either side of the drainage plane. The values of the stresses defining these lines are given by the following expressions:

$$\theta=0 \quad \sigma_y = -\frac{wy}{\sin^2\alpha}\Big[(\gamma\sin^2\alpha-\eta\sin^2\alpha-\cos^2\alpha)+ +\eta k\sin(\alpha-\beta)[\sin(\alpha-\beta)-n\cos\beta\sin\alpha]\Big]$$

$$\theta=\beta \quad \sigma_y = -\frac{wy}{\sin^2\alpha}\Big[\gamma\frac{\sin\alpha\sin(\alpha-\beta)}{\cos\beta}-\cos^2\alpha\times \times\left(1-\frac{2\tan\beta}{\tan\alpha}\right)-\eta n\sin^2\alpha-2\eta k\sin(\alpha-\beta)\times \times\frac{\tan\beta}{\tan\alpha}[\sin(\alpha-\beta)-n\cos\beta\sin\alpha]\Big] \qquad \ldots(8)$$

$$\theta=\alpha \quad \sigma_y = -\frac{wy}{\sin^2\alpha}[\cos^2\alpha+\eta k\sin(\alpha-\beta)\cos\alpha\sin\beta- -\eta nk\sin\alpha\cos\alpha\sin\beta\cos\beta]$$

and

$$\theta=0 \quad \tau_{xy}=0$$

$$\theta=\beta \quad \tau_{xy} = -\frac{wy\tan\beta}{\sin^2\alpha}[\cos^2\alpha-\eta k\sin^2(\alpha-\beta)\times \times(1-n\cos^2\beta)+\eta nk\sin\beta\cos\beta\times \times\sin(\alpha-\beta)\cos(\alpha-\beta)]$$

$$\theta=\alpha \quad \tau_{xy} = -wy\cot\alpha- -\eta k\frac{wy\sin(\alpha-\beta)}{\sin\alpha}\sin^2\beta\left[\frac{1-n\cos^2\beta}{\sin\beta}-\frac{n\cos\beta}{\tan(\alpha-\beta)}\right]$$

For comparison with the example shown in Fig. 5 (which is a specific case of the above example with $n=0$) a plot of vertical stress for a set of definite numerical values is shown in Fig. 7. Here:

$\alpha=40°$ $\qquad$ $\beta=20°$

$\gamma=2{\cdot}42w$ $\qquad$ $\eta=100\%$

The distribution of stress is shown for different values of Poisson's ratio and for comparison the stresses obtained by extending Brahtz' theory are shown. A line calculated on the assumption of a linear distribution of vertical stress is also shown. This is arrived at by assuming that pore pressure acts as a resultant force through the centre of gravity of the pressure diagram and then considering moments of the three forces—weight, horizontal thrust and uplift—about the horizontal section. It will be noted that whereas there is a considerable discrepancy between the extension of Brahtz' theory and the true stresses, the linear assumption gives a remarkably good approximation. This result, intuitively assumed and used by many designers, has so far lacked any theoretical support; all analyses of stress distribution in gravity dams leading to an approximately linear distribution of stress having been based on purely external application of forces.

†The solution outlined below forms the subject of a recent paper by one of the authors.[7]

From numerous computations it has been found that the above conclusion is quite generally true within the usual range of values of the physical constants and assumptions of pressure distribution which are applicable to a concrete dam. For example, for a dam with the profile and physical properties assumed for Fig. 7, the linear assumption gives a maximum error of approximately 4% of wy at the upstream face when β is approximately 20° and $\nu=0$. This error decreases to zero as β tends to the values α or zero. For greater values of Poisson's ratio the errors incurred by the linear assumption increase, but within the range applicable to concrete, say zero to 0·20, they are never of serious magnitude. As an illustration, Fig. 8 shows how the errors in σ_y at the upstream and downstream faces vary with the position of the drainage plane if Poisson's ratio is assumed to have the value 0·16. The apparent maximum error of 0·189wy at the upstream face when $\beta=0$ is of theoretical interest only, and for a practical case β is unlikely to be less than about 5° for an apex angle of 40°. For this value, the error at the upstream face is 0·167wy or 6·8 lb. per sq. in. for a section at a depth of 100 ft, hardly a serious error in comparison with the degree of approximation used in gravity-dam analysis.

Fig. 9 gives an example of the distribution of shear stress using equations (9), and also by assuming a linear distribution. In this case, and in general, a linear assumption incurs errors of greater magnitude than in the case of vertical stresses. The error has a maximum value when $\beta=\frac{1}{2}\alpha$, decreasing to zero as β tends to zero or to α. For the example shown the linear assumption underestimates the maximum shear stress by 13% when $\nu=0$, but the error decreases with increasing values of ν, contrary to the effect observed with vertical stresses.

Distribution of Stresses in Cylindrical Sections

We consider now the distribution of stress in a porous cylinder or pipe of internal radius a and external radius b, subjected to three different systems of pressure distribution, namely:

1. radial percolation of a fluid from pressure p_a at the internal boundary a, to zero at the external boundary b
2. radial percolation of a fluid from pressure p_a at the internal boundary a, to zero at some concentric drainage boundary at radius c, where $a<c<b$
3. a static pressure p_a in the region $a<r<c$ caused by the presence of an impermeable membrane at radius c.

The first case considered is a practical problem which occurs frequently, but is seldom treated as a pore-pressure problem. The two remaining cases, in the simple form in which they are presented here, are academic in nature, but they may be regarded as a

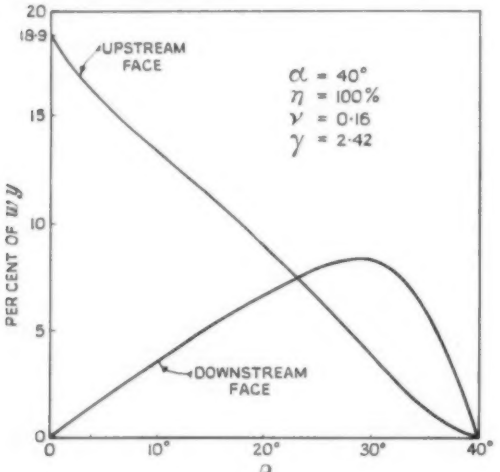


FIG. 8

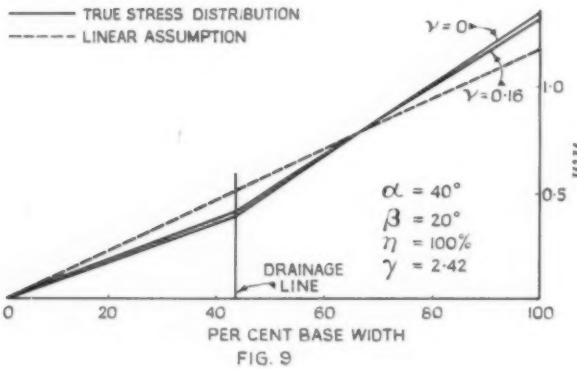
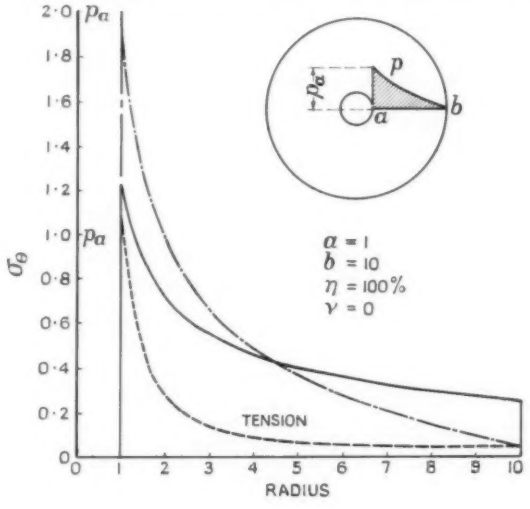


FIG. 9



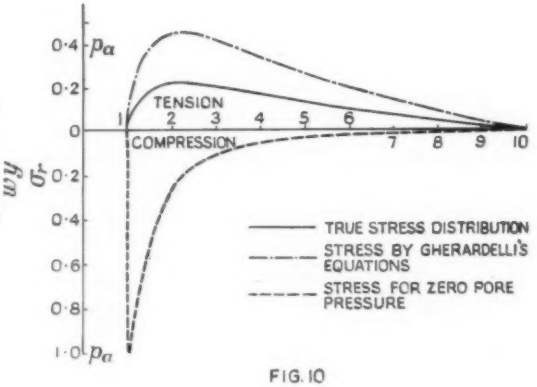


FIG. 10

first step towards a solution of more difficult problems of practical interest. For example, the second problem approximates to the case of a concrete-lined tunnel through fissured rock which has the same elastic properties as concrete but much greater permeability. The third problem simulates the hydraulic conditions in a pipe or tunnel with a thin steel lining, further lined by concrete.

General Theory for Cylindrical Sections[5]

Since the cases to be considered are in all respects symmetrical, it is convenient to use a simpler stress function than that used in the dam problem. The equilibrium equation in polar co-ordinates for a symmetrical system is:

$$\frac{d\sigma_r}{dr}+\frac{\sigma_r-\sigma_\theta}{r}+R=0$$

where R is the body force per unit volume, which in this case, due to pore pressure and neglecting the effect of gravity, is equal to $-\eta(dp)/(dr)$. The equation of equilibrium thus becomes:

$$\frac{d\sigma_r}{dr}+\frac{\sigma_r-\sigma_\theta}{r}-\eta\frac{dp}{dr}=0 \qquad \ldots(10)$$

Equation (10) is automatically satisfied if the stresses are defined in terms of a stress function F in the following manner:

$$\sigma_r=\frac{F}{r}$$
$$\sigma_\theta=\frac{dF}{dr}-\eta r\frac{dp}{dr} \qquad \ldots(11)$$

The condition for compatibility of strain in a symmetrical system, expressed in terms of the radial and tangential strain components, is:†

$$\epsilon_\theta-\epsilon_r+r\frac{d\epsilon_\theta}{dr}=0 \qquad \ldots(12)$$

If the strain components ϵ_r and ϵ_θ are now expressed in terms of the stresses σ_r and σ_θ in accordance with the usual plane strain relations derived from Hooke's law, then using equations (11) it can be shown that the stress function F must satisfy the following relation:

$$r\frac{d^2F}{dr^2}+\frac{dF}{dr}-\frac{F}{r}=\frac{\eta}{1-\nu}r\frac{dp}{dr}+\eta r\nabla^2 p \qquad \ldots(13)$$

Equation (13) is the governing equation, the general solution of which is made possible by the use of the substitution $r=ae^t$. The solutions for the three cases of pressure distribution already described will now be considered in turn.

Case 1.

The radial percolation of the fluid, assuming that it obeys Darcy's law and follows the relation $\nabla^2 p=0$, leads to a pressure distribution in the material defined by:

$$p=p_a\frac{\log b/r}{\log b/a} \qquad \ldots(14)$$

Inserting this value in equation (13) and using the substitution $r=ae^t$ the general solution is found to be

$$F=C_1 r+C_2\frac{1}{r}+\frac{\eta p_a}{2(1-\nu)\log a/b}r\log r$$

C_1 and C_2 are arbitrary constants which must be determined from the boundary conditions:

$$\sigma_r=-p_a(1-\eta) \quad \text{at } r=a$$
$$\sigma_r=0 \quad \text{at } r=b \qquad \ldots(15)$$

Using equations (11) the stress components are found to be:

$$\sigma_r=\frac{p_a a^2}{b^2-a^2}\left(1-\frac{b^2}{r^2}\right)\left[\frac{\eta}{2(1-\nu)}+1-\eta\right]+\frac{\eta p_a}{2(1-\nu)}\frac{\log b/r}{\log b/a} \qquad \ldots(16)$$

$$\sigma_\theta=\frac{p_a a^2}{b^2-a^2}\left(1+\frac{b^2}{r^2}\right)\left[\frac{\eta}{2(1-\nu)}+1-\eta\right]+\frac{\eta p_a}{2(1-\nu)}\frac{\log b/r}{\log b/a}+\frac{\eta p_a}{2\log b/a}\frac{(1-2\nu)}{(1-\nu)}$$

It is interesting to compare this result with that found by applying Brahtz' theory to the problem as was done by Gherardelli[6]. The distribution of stress in a non-porous cylinder subjected to an internal pressure p_a is defined by the well-known Lamé formulae:†

$$\sigma_r=\frac{p_a a^2}{b^2-a^2}\left(1-\frac{b^2}{r^2}\right)$$
$$\sigma_\theta=\frac{p_a a^2}{b^2-a^2}\left(1+\frac{b^2}{r^2}\right)$$

(Easily derived from equations (16) by making $\eta=0$) As the pressure distribution is defined by equation (14) and obeys the relation $\nabla^2 p=0$ throughout the region of the porous material, it follows by Brahtz' theory that the stresses should be:

$$\sigma_r=\frac{p_a a^2}{b^2-a^2}\left(1-\frac{b^2}{r^2}\right)+\eta p_a\frac{\log b/r}{\log b/a}$$
$$\sigma_\theta=\frac{p_a a^2}{b^2-a^2}\left(1+\frac{b^2}{r^2}\right)+\eta p_a\frac{\log b/r}{\log b/a} \qquad \ldots(17)$$

The discrepancy between expressions (16) and (17) is at once apparent and is illustrated in Fig. 10 where, for particular values of a and b, σ_r and σ_θ are plotted according to each set of expressions.

Case 2.

The distribution of pressure for concentric drainage at radius c is defined by:

$$p=p_a\frac{\log c/r}{\log c/a} \quad \text{in the region } a<r<c$$
$$p=0 \quad \text{in the region } c<r<b$$

Using again the substitution $r=ae^t$,

$$\nabla^2 p=\frac{d^2p}{dr^2}+\frac{1}{r}\frac{dp}{dr}=\frac{1}{a^2e^{2t}}\frac{d^2p}{dt^2}$$

and the governing equation (13) becomes:

$$\frac{d^2F}{dt^2}-F=\frac{\eta}{1-\nu}ae^t\frac{dp}{dt}+\eta ae^t\frac{d^2p}{dt^2}$$

The solution of this equation is effected by the Laplace

†See reference 4, section 30.

†See reference 4, section 26.

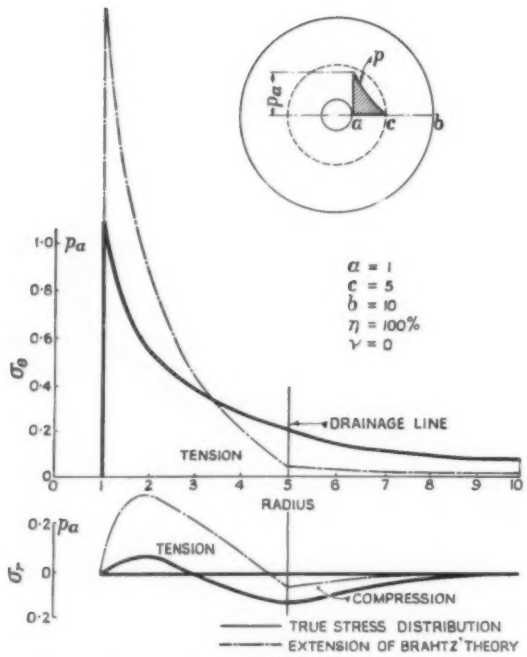


FIG. 11

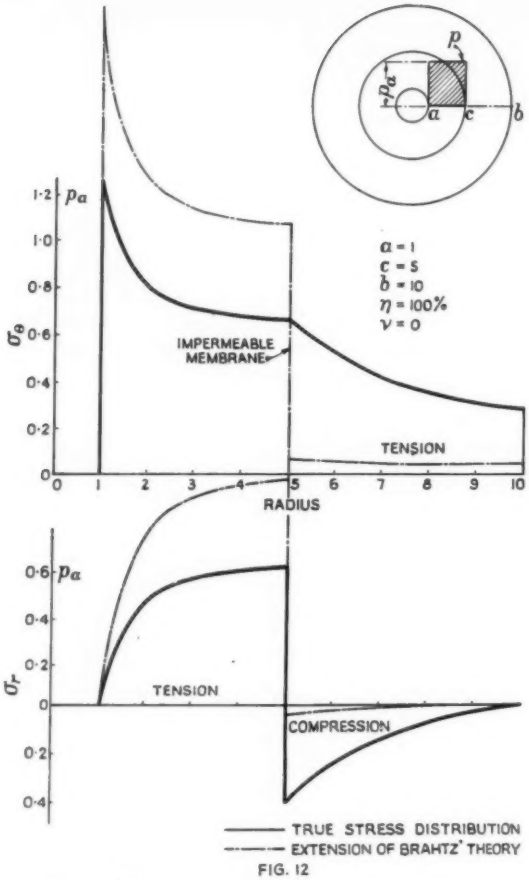


FIG. 12

transformation method to give:

$$F=A\frac{r}{a}+B\frac{a}{r}-\frac{\eta a p_a}{4(1-\nu)\log c/a}\left(\frac{a}{r}+2\frac{r}{a}\log\frac{r}{a}-\frac{r}{a}\right)+$$

$$+\frac{\eta a p_a}{4\log c/a}\frac{c}{a}\left[\frac{1-2\nu}{1-\nu}\frac{r}{c}+\frac{2}{1-\nu}\frac{r}{c}\log\frac{r}{c}-\frac{1-2\nu}{1-\nu}\frac{c}{r}\right]H(r-c)$$

where $H(r-c)=0$ for $a<r<c$
$H(r-c)=1$ for $c<r<b$

and A and B are arbitrary constants.

Substituting this value of F into the stress expressions (11) and using the boundary conditions which are identical to those in case 1 to find the arbitrary constants A and B, the stress components are found to be:

$$\sigma_r=\frac{p_a a^2}{b^2-a^2}(1-b^2/r^2)\left[1-\eta+\frac{\eta}{2(1-\nu)}-\frac{\eta(1-2\nu)}{4(1-\nu)}\times\right.$$
$$\left.\times\frac{(1-c^2/b^2)}{\log c/a}\right]+\frac{\eta p_a}{4(1-\nu)\log c/a}[2\log c/r-(1-2\nu)\times$$
$$\times(1-c^2/b^2)]+\frac{\eta p_a}{4(1-\nu)\log c/a}[2\log r/c+(1-2\nu)\times$$
$$\times(1-c^2/r^2)]H(r-c)$$
$$\ldots(18)$$
$$\sigma_\theta=\frac{p_a a^2}{b^2-a^2}(1+b^2/r^2)\left[1-\eta+\frac{\eta}{2(1-\nu)}-\frac{\eta(1-2\nu)}{4(1-\nu)}\times\right.$$
$$\left.\times\frac{(1-c^2/b^2)}{\log c/a}\right]+\frac{\eta p_a}{4(1-\nu)\log c/a}[2(\log c/r-1)-$$
$$-(1-2\nu)(1-c^2/b^2)]+\frac{\eta p_a}{4(1-\nu)\log c/a}\times$$
$$\times[2(\log r/c+1)+(1-2\nu)(1+c^2/r^2)]H(r-c)+\eta r\frac{dp}{dr}$$

where $\eta r\dfrac{dp}{dr}=-\dfrac{\eta p_a}{\log c/a}$ for $a<r<c$

$\eta r\dfrac{dp}{dr}=0$ for $c<r<b$

The stresses given by expressions (18) are illustrated in Fig. 11, with the results obtained by extending Brahtz' theory given for comparison.

To apply expressions (18) to the case of a tunnel through fissured rock, it is necessary to assume that radius b is large in comparison to radii a and c. This leads to the following expressions:

$$\sigma_r=-\frac{p_a a^2}{r^2}\left[1-\eta+\frac{\eta}{2(1-\nu)}-\frac{\eta(1-2\nu)}{4(1-\nu)\log c/a}\right]+$$
$$+\frac{\eta p_a}{4(1-\nu)\log c/a}[2\log c/r-(1-2\nu)]+$$
$$+\frac{\eta p_a}{4(1-\nu)\log c/a}[2\log r/c+(1-2\nu)\times$$
$$\times(1-c^2/r^2)]H(r-c)$$
$$\ldots(19)$$

$$\sigma_\theta=\frac{p_a a^2}{r^2}\left[1-\eta+\frac{\eta}{2(1-\nu)}-\frac{\eta(1-2\nu)}{4(1-\nu)\log c/a}\right]+$$
$$+\frac{\eta p_a}{4(1-\nu)\log c/a}[2(\log c/r-1)-(1-2\nu)]+$$
$$+\frac{\eta p_a}{4(1-\nu)\log c/a}[2(\log r/c+1)+(1-2\nu)\times$$
$$\times(1+c^2/r^2)]H(r-c)-\eta r\frac{dp}{dr}$$

Case 3.

The distribution of pressure for an impermeable membrane at radius c is defined by:

$$p=p_a \quad \text{in the region } a<r<c$$
$$p=0 \quad \text{in the region } c<r<b$$

Using the substitution $r=ae^t$, the governing equation (13) becomes, as for case 2,

$$\frac{d^2F}{dt^2}-F=\frac{\eta}{1-\nu}ae^t\frac{dp}{dt}+\eta ae^t\frac{d^2p}{dt^2}$$

The solution is again effected by the Laplace transformation method giving, for this pressure distribution:

$$F=A\frac{r}{a}+B\frac{a}{r}-\eta\frac{p_a}{2}r\left[\frac{1}{1-\nu}+\frac{1-2\nu}{1-\nu}\frac{c^2}{r^2}\right]H(r-c)$$

Substituting F into the stress expressions (11), and using the boundary conditions (which are again the same as for case 1) to determine the arbitrary constants A and B, the stress components are found to be:

$$\sigma_r=\frac{p_a a^2}{b^2-a^2}\left(1-\frac{b^2}{r^2}\right)\left[1-\eta+\frac{\eta}{2(1-\nu)}\left(1+(1-2\nu)\frac{c^2}{b^2}\right)\right]+$$
$$+\frac{\eta p_a}{2(1-\nu)}\left[1+(1-2\nu)\frac{c^2}{b^2}\right]+\frac{\eta p_a}{2(1-\nu)}\times$$
$$\times\left[1-(1-2\nu)\frac{c^2}{r^2}\right]H(r-c) \qquad \ldots(20)$$

$$\sigma_\theta=\frac{p_a a^2}{b^2-a^2}\left(1+\frac{b^2}{r^2}\right)\left[1-\eta+\frac{\eta}{2(1-\nu)}\left(1+(1-2\nu)\frac{c^2}{b^2}\right)\right]+$$
$$+\frac{\eta p_a}{2(1-\nu)}\left[1+(1-2\nu)\frac{c^2}{b^2}\right]-\frac{\eta p_a}{2(1-\nu)}\times$$
$$\times\left[1-(1-2\nu)\frac{c^2}{r^2}\right]H(r-c)$$

The stresses given by equations (20) are illustrated in Fig. 12.

Conclusion

The examples chosen to apply the method of analysis based on considering pore-water pressure as a body force, have shown that little additional difficulty is introduced into the solution of problems which are amenable to analysis when loaded on the boundaries only. Limitations of simplified theories have been amply demonstrated and it is hoped that the results obtained will be of use in the solution of many engineering problems. In the case of dams, the approximate truth of the linear-stress-distribution assumption shows that such criteria as Levy's rule,[9] which did not recognise the beneficial effects of internal drainage, are not to be trusted. Incidentally the reason for the stability of many dams, based on uplift area factors as low as 20 to 40%, is undoubtedly due in part to the reduction of uplift caused by drainage and variations of concrete permeability within the dam, and not to the fallacy of the experiments showing much higher area factors.

The analyses of stresses in thick cylinders show that in general the stresses are smaller than those which would have been deduced by simplified hypothesis.

Acknowledgement

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A Glen Moriston Film

We recently had the pleasure of seeing a particularly satisfying documentary film sponsored by Mitchell Construction Company to show the part they had played in the execution of the North of Scotland Hydro-Electric Board's Glen Moriston scheme. This scheme has many points of interest. It comprises a main dam, 2,165 ft. long by 118 ft. high, at Loch Cluanie, and a smaller dam, 1,745 ft. long by 61 ft. high at Loch Loyne, several miles of 12 ft. diameter concrete-lined tunnels, and the first major underground power station in Scotland. Both dams were built with precast facing blocks, obviating the need for shuttering, and were the first dams in Britain constructed with Trief cement. This type of cement employs 70% of wet-ground blast-furnace slag, securing an important saving in cement and presenting many technical advantages. In the tunnels the rate of advance was exceptional, and Mitchell Construction Company claimed successively British, European and world tunnelling records. The underground station at Ceannacroc has a capacity of 20 MW, and when the entire catchment has been developed the scheme will have an installed capacity of 65 MW and an annual production of 213 million kWh.

The film traces the progress of the contract from the preliminary surveys to the commencement of generation. Mitchell Construction Company were the civil-engineering contractors for the scheme, and the consulting engineers were Sir William Halcrow and Partners.

Fig. 1. Kaieteur Gorge from the top of the Falls

Investigations on the Potaro

M. M. de Leva, A.S.C.E., A.I.E.E., and F. J. Bartholomew, F.A.I.E.E., give an outline of a series of investigations they have carried out which indicates the existence of a large hydro-electric prospect on the Potaro River in British Guiana

THIS article is the outcome of a series of investigations that the writers undertook in 1953-4 on behalf of British Guiana Consolidated Goldfields in relation to the works for a 2,000 h.p. hydro-electric plant at Tumatumari on the Potaro. Studies were undertaken also on the Demerara River, which is more accessible than the Potaro, but has nowhere near its potentialities.

In the early maps the entire north-east of the South American continent between the Amazon in the south and the Orinoco in the north is shown under the name "Guiana," an Amerindian word that means "Land of Waters," and the name is most appropriate, in view of the heavy rainfall and large rivers that characterise this area. The expanding European nations of the 17th and 18th centuries subdivided Guiana into five different sections: the Portuguese, that is now northern Brazil or Amazonia; the French; the Dutch, known better as Surinam; the British, which was part of the original Dutch colony; and the Spanish, now the eastern part of Venezuela. A large part of the Guianas is still only partially known, and the dense tropical rain forests render exploration costly, difficult and hazardous. This explains why, in a century that is constantly in search of more power, very little has been done to tap the multiple resources of the rivers that intersect the region; in fact, even parts of the courses of many of these rivers are known only approximately, with vague indication of flows and drops in elevation.

In British Guiana, according to the limited measurements available, it appears that the Potaro River basin receives close to the highest rainfalls of the north-east of the Continent, and here one should look for important hydro-electric developments. This river is a tributary of the Essequibo, the largest in British Guiana, and has its source in the Ayanganna mountains, a high plateau type of range, with flat-topped peaks that reach about 6,000 ft., and form the backbone of the Pakaraima range, separating the three Guianas, British, Venezuela, and Brazil; Mount Roraima, elevation 9,094 ft., the highest point in the area, is part of this range just outside the Potaro watershed. The course of the Potaro has a length of 190 miles. For the first 55 miles it passes through a characteristic geological formation that takes its name from Mount Roraima, and consists of terraced sedimentaries, with a practically undisturbed horizontal stratification. All the higher reaches consist of an open Savanna type of country, and only when the course drops to the 4,000-3,000 ft. level does it enter the dense tropical forests.

The first major waterfall is encountered at Yatuk, immediately below the confluence with a large tributary, the Kopinang. From here the river meanders in a high-elevation peneplain until it drops vertically for 741 ft. in the spectacular Kaieteur Falls (Fig. 3). The next 15 miles are through a wide and high-walled gorge that ends just above some small falls or rapids known as Amatuk Falls. Out of the gorge and below Amatuk the Potaro has a very irregular course beset by many small falls and rapids caused by relatively recent outcrops or dykes of dolerite. The last and best known of these is Tumatumari Falls, the foot of which is at approximately 80 ft. elevation, and corresponds to the level of the Essequibo at its confluence with the Potaro, some five miles further down.

Another river that has its source in the Ayanganna mountains close to the Potaro is the Kuribrong, that, after a course of some 92 miles, becomes a tributary of the Potaro, some ten miles above Tumatumari. This river also leaves the escarpment of the Pakaraima formation with a waterfall, known as Amaila Falls, but here there is neither the water volume nor the vertical grandeur of Kaieteur.

Rainfall data covering the basin of the Potaro are

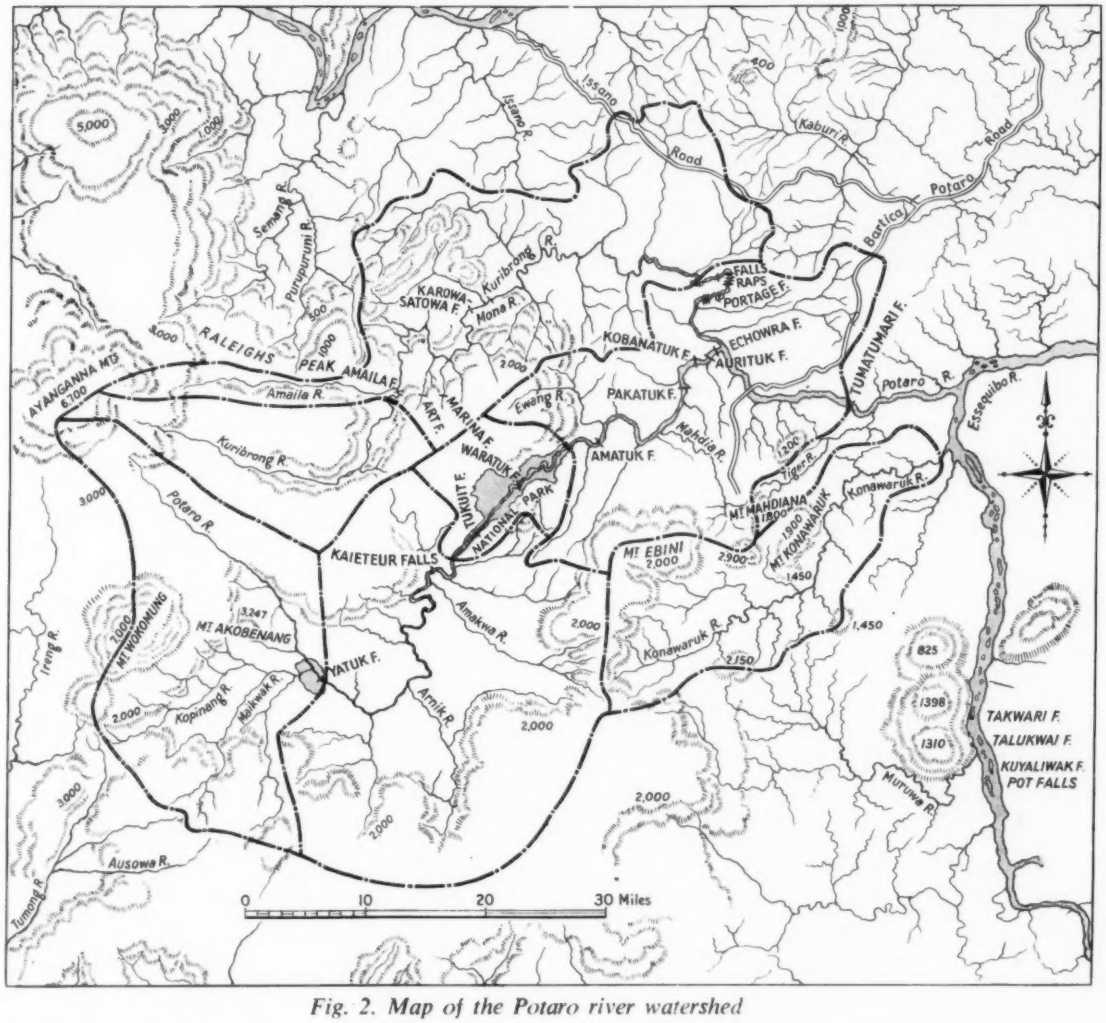


Fig. 2. Map of the Potaro river watershed

Fig. 3. Aerial view of Kaieteur Falls

very limited and sketchy, although two stations below Amatuk have records that go back some 30 years. To make any sort of useful study it is necessary to extrapolate what information is available regarding the basin itself, and check this by comparison with a few scattered rainfall stations sited at those rare inhabited locations where conditions are generally similar enough to permit comparisons. Here following is a tabulation of some recent measurements, together with long-time averages for a few stations:—

	Average in.	1953 in.
Mahdia (Potaro)	149	169
Tumatumari (Potaro) ..	113·2	139
Kamakusa (Mazaruni) ..	134	193*
Kaieteur	—	185

Long-term flow records of the Potaro exist only for Tumatumari, and they now cover some 16 to 17 years. They were taken during two different periods, one from 1921 to 1930, and the second from 1946 to 1954, when the construction of the small hydro-electric plant for British Guiana Consolidated Goldfields' local use interfered with the calibration of the gauging system.

The two sets of measurements have different zero points, and the correlation between the two records is imperfect, but the difference is not such as to impair conclusions of a general nature. Fig. 4 gives a flow-frequency curve for Tumatumari. Main data are as follows:—

Watershed area	2,400 sq. miles
Maximum flow (1947) ..	110,000 cusecs (estimated)
Minimum flow (1947) ..	750 cusecs*
Average flow (1946-1953)	17,250 cusecs

In 1949 a gauging station was set up at the top of Kaieteur Falls, and soon from here there will be enough accurate data to justify reasonably long-term predictions. Nothing, on the other hand, is known regarding rainfall or river flows for the Kuribrong basin.

The rainfall in British Guiana has typically tropical characteristics with a season of extremely heavy precipitation covering the months of May, June, July; a relatively dry period lasting until November-December; a second less intensive rainy season until February, followed by a second dry period. It should be stated at once that this is a theoretical distribution; the saying is that there are only two seasons in British Guiana, a "rainy" season and a "wet" season, and in actual fact there is a very great irregularity and variation from year to year. The June-July rains are constant and have never been known to miss; but the intermediate dry periods often do not occur at all as such, and only appear as reductions in precipitation. Once in a while there are no December-January rains, and a bad drought settles over the entire country; it

* This is the highest precipitation recorded at this station.

* The minimum flow in 1926 was considerably less, but reliable figures are not available

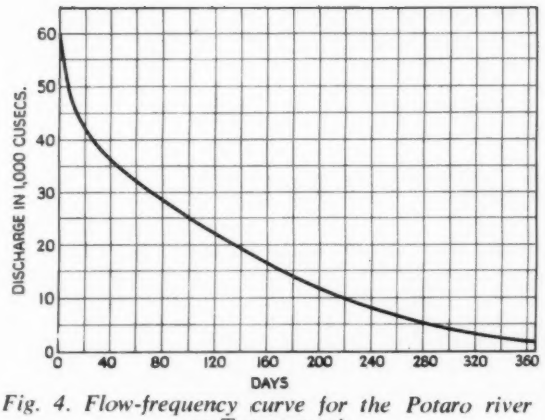


Fig. 4. Flow-frequency curve for the Potaro river at Tumatumari

starts in September-October and goes on practically uninterrupted until the April-May rains bring relief. Under these conditions the entire countryside becomes parched, the forests catch fire, and the Potaro drops to a flow of a few hundred cusecs. There is an undocumented mention of 250 cusecs or less at Tumatumari made by people who have been in the area some decades ago; the extreme low is liable to last approximately one month. Within recorded memory droughts such as these have occurred in 1901, 1912, 1926, 1940 and 1947. Those of 1912 and 1926 apparently were the most severe.

The geological formation of the Potaro watershed is typical of most of the southern part of British Guiana. The lower reaches consist of an igneous base that is covered in places by white sands, and in others by a layer of laterite, some 6 to 10 ft. deep, that is a superficial decaying of the underlying igneous formation. At many points dykes of very hard dolerite have thrust themselves through the igneous formation, causing a series of low hills and undulations, and creating the frequent rapids and small falls that interrupt the course of most of the river, impeding navigation and rendering cumbersome portages necessary. Typical examples of these are the falls at Amatuk and Tumatumari.

From this low-lying section that rarely exceeds an elevation of 400 to 500 ft., rise the vertical cliffs of the Roraima formation. The first man to study this geology was C. B. Brown, who around 1875 made a series of journeys into the interior, and who was the discoverer of Kaieteur. His book, long out of print, still represents a first-class reference for most of the country.

The main characteristics of this Roraima formation are a series of practically horizonal layers of miscellaneous sandstones, conglomerates, and breccias, conglomerates with gabbro sills appearing at various points. Its geological history and age are still something of a question mark, as, surprisingly, despite typically sedimentary characteristics, so far nobody has found any fossils and some authors believe that this sedimentation is so ancient as to be before major organic life.

For hydro-electric developments the geological aspect appears favourable, because the very regular and horizontal stratifications lend themselves to easy investigation, with less probability of costly contingencies. The rocks in general are good for construction and range from extremely hard dolerites that have proved excellent stone-masonry material, but also very hard and tenacious so that drilling and blasting come expensive, to sandstones. The specific gravity of the dolerites is around 3·0, and their weight and compactness can be considered an asset for any gravity structure. The sandstones usually have just sufficient consistency for most types of work, but cannot be recommended as wearing surfaces of spillways.

Hydro-Electric Potentiality

Fig. 6 gives an approximate profile of the Potaro River, leaving out such higher sections as are of little or no interest from a hydro-electric point of view, and where flows are liable to have torrential characteristics. In this sketch the horizontal lengths shown are air distances, and do not take into account the meandering of the river.

The profile shows that the first major falls are at Yatuk, at an elevation of approximately 1,650 ft. Few people, apart from local Amerindians, have ever been near the site, and no measurements or details are available. Mr. M. M. de Leva had occasion to make

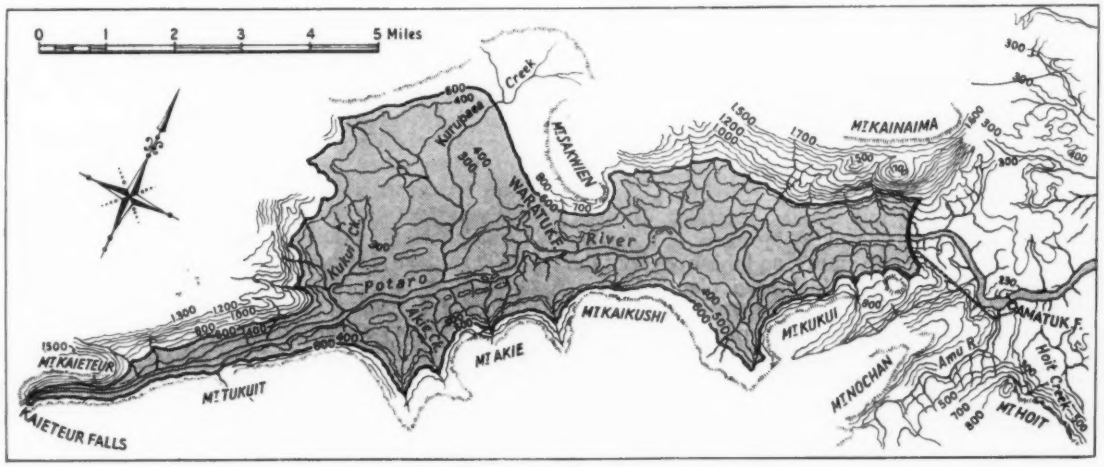


Fig. 5. Storage area in the Kaieteur Gorge

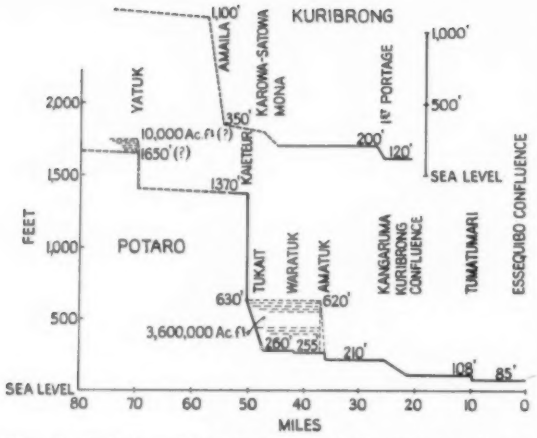


Fig. 6. Profiles of the Potaro and Kuribrong rivers

a quick examination from the air, taking some photographs, and these show that there definitely is the possibility of developing the location for the generation of power. A tentative estimate of the head puts it at 250 ft., and a cut where the river passes between two hills might be the location for a dam, creating an appreciable storage area behind. The following figures are given purely as an indication regarding the order of size of plant that could be envisaged:—

Head	250 ft.
Average flow	3,000 cusecs
Available storage	10,000 acre ft.
Power	5,000 to 10,000 h.p.

The distance from Georgetown and from all other potential users makes it improbable that this plant will be constructed for many years, unless exploration of the area brings forth some important mining project.

The major outstanding feature of the profile is the vertical drop at Kaieteur, together with the storage possibilities below. The average flow here is of the order of 8,000 cusecs, but low minima of 600 have already been recorded, and worse droughts can easily occur at frequent intervals.

The terrain above the falls is completely open with low undulations that make storage virtually impossible at the site. Perhaps the flow could be regulated by a series of small dams on the higher reaches and on the tributaries so as to obtain a sufficient cumulative effect, but the distances involved and the undeveloped nature of the region makes this totally impracticable.

Instead, immediately below the falls the Potaro enters a gorge and drops in a series of rapids to a level of approximately 260 ft. This gorge (Fig. 1) has an average width of 1½ miles, and a length of 15 miles, while two miles outside the gorge there is a further drop of about 45 ft. at Amatuk, and this opens out the possibility of a relatively high-head plant of major proportions.

A large dam built at the mouth of the gorge to an elevation of approximately 620 ft. would create an artificial lake with a content of approximately 3,600,000 acre ft. of water. The height of the dam itself would be some 360 ft. above the river bed, and this, plus the drop at Amatuk, would give a total head of some 410 ft. Taking into account water losses and average drawdown, there would remain available a net head of approximately 350-360 ft.

The elevation of 620 ft. for the crest is determined mainly by the height of the walls of the gorge at the outlet and at the bottom of Kaieteur Falls. This would make the fullest use of the existing rock buttresses, while in no way impairing the majestic beauty of Kaieteur. This is an important consideration, because the falls have been declared a national monument of British Guiana, and any attempt to interfere with them would certainly run into considerable local opposition.

An underground power house close to the dam appears equally feasible, and a tunnel tailrace at low elevation could discharge the water below Amatuk.

A storage of this nature would regulate the flow of the Potaro on an annual basis, and close to 300,000 h.p. would become available.

At this preliminary stage it is impossible to predict what type of dam is most suitable for this site. Single or multiple concrete arches or rockfill gravity have all got points in their favour, and only a very thorough economic investigation can indicate which is the most suitable, especially in view of the very large structure involved, and transportation difficulties to the site requiring many miles of new roads through the forest.

A complete regulation of the Potaro at this point would also create considerable advantages downstream. For one thing, the power potential at Tumatumari, based on a year-round availability, would become five or six times greater; secondly, the navigation on the Essequibo would be improved because now during the dry seasons the water can become very low and the passage of some of the rapids hazardous for small craft and impossible for the larger barges.

The Kaieteur hydro-electric plant by no means exhausts the hydro-electric potentialities of British Guiana. To the west there are other rivers of importance, but they are even less known. One in particular, the Mazaruni, has a greater watershed and flows are proportionately larger; its source is also in the Pakaraima mountains, whence it comes down through gorges with waterfalls of which very little is known.

It is to be hoped that during the coming decades this practically unknown area will be opened up by projects of this nature, and its resources made available to the progress of civilisation.

Contract for Underground Cable. The Auckland Electric Power Board, North Island, New Zealand, have decided on a scheme of expansion in which an important factor will be the provision of an additional power link with the Penrose substation of the State Hydro-Electric Department and the Board's substation at Quay. For this power connection the Board decided to adopt a 110 kV underground cable system and have now awarded the contract to British Insulated Callender's Cables Limited. The cable will be 22,500 yards in length and will be of the 110 kV three-core impregnated pressure type having copper conductors of 0·2 sq. in. area and an aluminium sheathing. Supervisory and jointing services connected with the installation of the cable will be provided by the staff of B.I.C.C. Construction (N.Z.) Limited. The total value of this contract will amount to some £186,000 and is the most important for high-voltage cable ever placed in New Zealand. It is expected that it will be completed towards the end of 1958.

Cavitation in Turbomachines

In this article, which follows a contribution by Dipl. Eng. F. A. L. Winternitz on the same subject, K. K. SHALNEV, of the Institute of Mechanics, Academy of Sciences, U.S.S.R., places further data at the disposal of water-turbine designers

PART ONE

THE article by Dipl. Eng. F. A. L. Winternitz, containing a survey of modern information on cavitation in turbines and pumps, was published in the September, October and November, 1957, issues of WATER POWER. The problem of cavitation is of such importance for the further development of hydraulic design that publication of Winternitz's survey is particularly welcome, as it simplifies the work of designers and engineers who are making analogous cavitation investigations. The present author's contribution, published under the same title, should be regarded as being in the nature of a supplement to the article by Winternitz.

Cavitation terminology and trends in Cavitation Research

In his systematic analysis of the information available on cavitation the author soon realised the necessity to establish equivalent technical terms. Changes in concepts have been so rapid in recent years that it was necessary to establish some of the basic notions in appropriate terms. This terminology, it is realised, will continue to develop, but in the way it is presented here it greatly simplified the writer's work.

We will call cavitation (the term, suggested by Froude[1,2] in 1895) the appearance of a discontinuity in the moving liquid, including the oscillations of its particles, when the pressure in the rupture region becomes approximately equal to the sum of partial pressures of vapour and gases dissolved in liquid. The expression "cavity, filled with vapour and gas" is not used in this determination, because of the fact that the liquid in the rupture region may be in two phases: in vapour state and in liquid state (of bubble structure)[3].

Region or zone of cavitation is that part of liquid space in which the phenomenon of cavitation takes place, including those parts of liquid space where bubbles are formed and collapse. Sometimes it is useful to limit our consideration of the cavitation area as having more definite boundaries; this condition is characterised by the term "cavitation length"[4].

Cavitation bubbles are elementary liquid ruptures; their average diameter varies from 0·1 mm. to 2 mm., and the cavities are combinations of bubbles of different sizes, segregated into compact masses[3].

A stationary cavity is a motionless cavity, the form of which is steady in terms of time.

A periodic cavity is a cavity which forms in the flow periodically and which is periodically carried away by

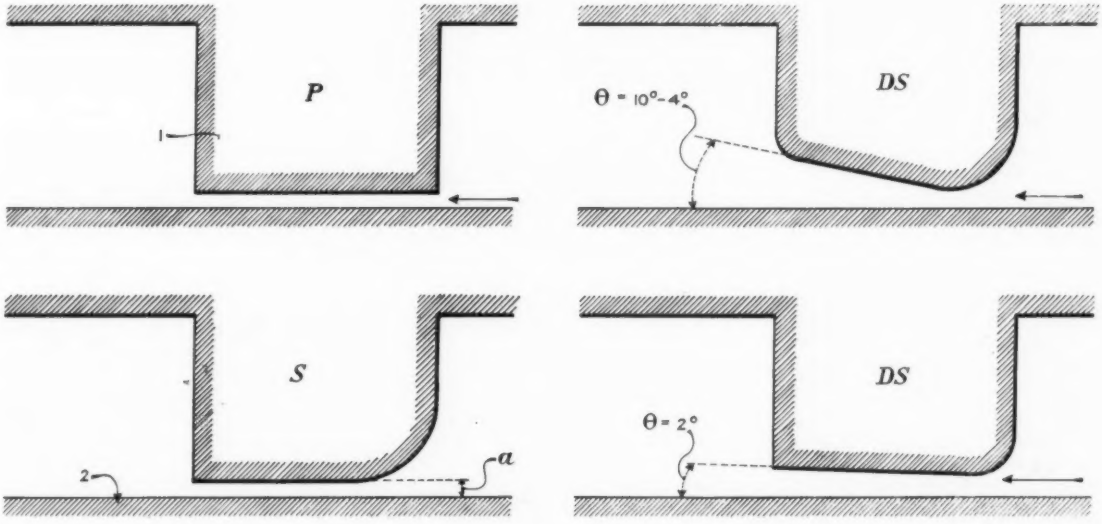


Fig. 1. Scheme of clearance models, cavitation of which was tested in the water tunnel: (1) contracting wall; (2) guiding wall

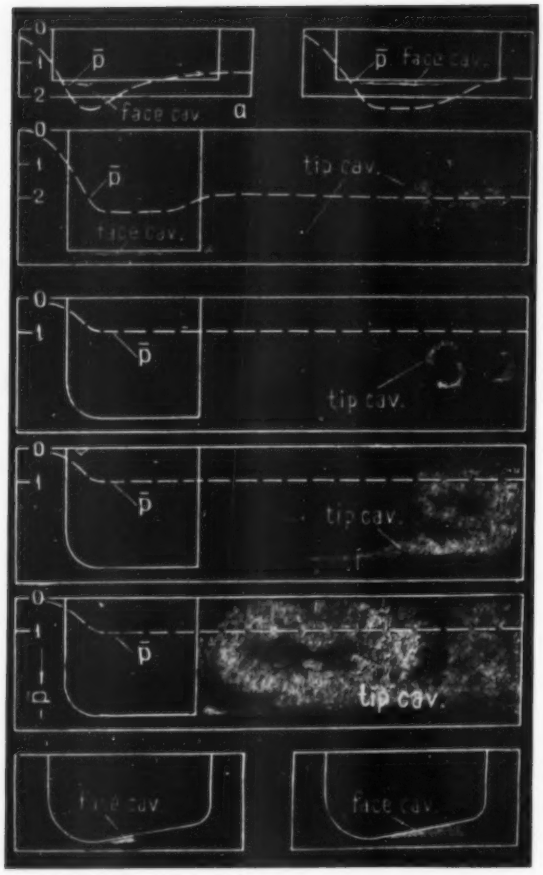


Fig. 2. Schemes of cavitation development in the various clearances at various values of H

the flow[5], or is a cavity which changes its form in the stationary liquid under the influence of variable pressure[6].

The expression "kind of cavitation" denotes a characteristic of cavitation on the basis of its location on a structural part.

As examples of "kinds" of cavitation[7] the axial-flow, variable-pitch turbine (the Kaplan) may be considered:—

Profile cavitation occurs on the blades as a result of the high incidence angle of relative flow, or as a result of the unsatisfactory design of the blade profile in its cylindrical coaxial section, i.e. when the profile is streamlined insufficiently.

Blade-clearance cavitation takes place on the blade face near the chamber wall or on the end near the hub if the blade profile is imperfectly shaped, i.e. incorrectly streamlined.

Blade-tip cavitation begins in a zone where two flows mix, i.e. the clearance flow below the suction side of the blade and the main flow along the blade. This kind of cavitation is related to the fact that the blade is finite and results from the non-uniform direction and varying velocities of the flow.

Roughness cavitation is caused by the rough surfaces of the blade or of the casing wall; this type of cavitation is easily observed in the early stages of the damage[8].

Blade-shoulder cavitation is caused by an unsatisfactorily-designed junction between the blade and the shoulder[9] by means of which it is bolted on to the runner hub.

As opposed to "kind" of cavitation, the term "type" of cavitation can also be usefully introduced, as indicating a cavitation characteristic brought about by the type of flow. For example, we will use the term "boundary cavitation" if the cavitation takes place in the thin boundary layer at the surface of streamlined profile; or "the burbling cavitation" if it takes place in a vortex stream behind insufficiently streamlined bodies. Cavitation of the wave type[10] forms in the loops of pressure in stationary waves[11].

It thus seems that we have to reject the term "vortex cavitation", since, to every appearance, all cavitation, even the boundary-type[12] is of vortex origin and this term will not therefore be a characteristic indication of either cavitation form.

The stage of cavitation or phase of its growth is characterised by the extension of the cavitation zone or by the value of the cavitation criterion. Any kind or type of cavitation may be encountered at various phases of its growth, depending upon the flow velocity and absolute pressure value. The type and the stage of cavitation are thus often confounded[13].

From the above definitions the connection between the stage of cavitation and the cavitation criterion (or, as it is often called, the cavitation coefficient), becomes clear. The only reason for preferring the term "cavitation coefficient" instead of "cavitation number", is that use of the word "number" is usually associated with the name of a scientist who established the relevant data connected with it.

Among the terms characterising cavitation results, only one, erosion, needs to be mentioned. The writer considers the term "erosion" to be preferable to "corrosion", for the causes are mainly of a mechanical nature.

Damage caused by cavitation is quite often not distinguished from that brought about by sediment carried along by the water flow and which can more properly be attributed to abrasion. In this respect the present writer agrees with Evance[14], who defined abrasion as a damage caused by the mechanical action of one solid on another. The pitting caused by abrasive action has the appearance of spoon-shaped hollows[15] having smooth surfaces. Abrasion is essentially a surface action which wears away the solid body, while the pitting caused by cavitation extends deeper into the metal, making it porous and spongy.

After this survey of the cavitation terminology we must give some consideration to trends in cavitation research, with particular reference to turbomachines. Here at least three opinions or trends can be distinguished. According to the first of these, cavitation must be studied on full-scale machines in practical operation. These operating conditions are then simulated in a laboratory. To investigate all the details of cavitation growth in a runner, the stroboscopic and acoustical methods are used.

According to the second opinion, cavitation investigations can best be conducted on models of the component parts, such as blade profiles, channels, clear-

ances, etc. The advocates of this method argue that it is impossible to find the real causes of cavitation phenomenon without using this step-by-step approach.

Finally, there are advocates of the third method who maintain that both these methods of investigation must be used in conjunction with one another. Such a task, however, requires the co-operation of many investigators.

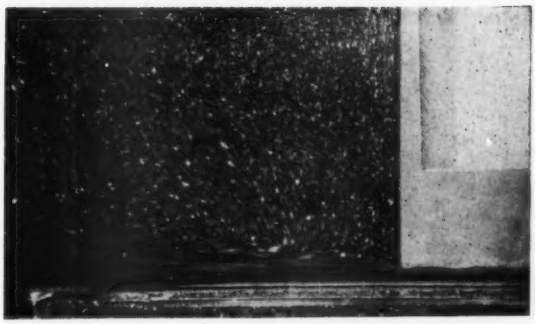
In the following it is proposed to give a short account of the results of certain cavitation studies which have been made and to attempt to establish the relation between various kinds of cavitation which exist in turbomachines and the different values of the machine cavitation coefficient σ.

It should be mentioned that local surface conditions, such as unevenness, which disturb the smoothness of the streaming, must be included in the elements to be investigated. These conditions may be a result of different causes—constructional design, machining methods or accidental damage.

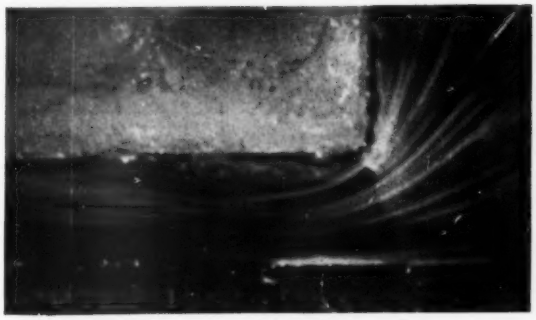
Cavitation at Blade Edges and Clearances

If we take a meridional cut through a blade and the adjacent walls of the runner casing, we shall have a clearance channel with a one-sided contraction of the flow at the inlet and a one-sided expansion at the outlet. In the middle part of the blade the size of this clearance channel usually varies in range $l/a=5$ to 20, where l is the length and a is the height of the channel.

The longitudinal profile of this channel may be of various contours. When the casing walls have spherical contours and the blades are turned accordingly, then, strictly speaking, the clearance channel will have curved walls parallel to each other. When the centres of these two spheres do not coincide, the channel profile becomes conoidal, that is, diffuser shaped. These conditions were reproduced in a model which was placed in the working chamber of a water tunnel[16, 17]. Each clearance model consisted of two parts—a contracting wall, representing the blade edge, and the guide wall, representing the wall of the runner casing (Fig. 1). The models were so sealed that the stream flowed only through the clearance channel between the contracting and guiding walls. To simplify the problem, the following versions were studied:— in (1) the clearance walls are parallel, the inlet edge of the contracting wall forming a straight angle—clearance type *P*; in (2) the clearance walls are again parallel, but the inlet edge of the contracting wall is rounded—clearance type *S*. In the two diagrams on the right of Fig. 1 the clearance walls diverge, the contracting wall being inclined to the guiding wall at the angle of

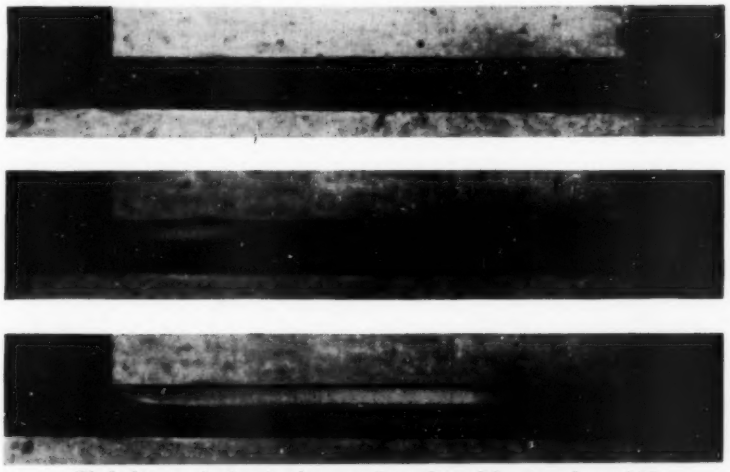


Flow round inlet edge in clearance **P**



Flow past model of clearance **S**

Fig. 4. Photographs of the surface stream through the clearance models without cavitation



Blade-face cavitation in clearance **P** *at three different values of* H_{cl}

Blade-face and blade-tip cavitation in clearance **P** *at low values of* H_{cl}

Fig. 3. Photographs of cavitation in clearance **P**

slope indicated by θ, thus forming a diffusion channel with the inlet edge rounded—clearance type *DS*.

Besides the cavitation investigations, the flow in open channels was studied, the visibility of flow being ensured by aluminium powder sprinkled on the water surface. Various types of clearances were investigated and the results are tabulated in Table I. In interpreting the test results, we shall use the following parameters:—

Flow-rate ratio of a clearance model

$$\mu=\frac{Q}{Q'}$$

where Q is the rate of flow through the model, measured in accordance with the water tunnel nozzle calibration by the weight method, and Q' is a theoretical rate of flow. The flow contraction coefficient at the clearance inlet is α; that is

$$\alpha=\frac{a_c}{a}$$

where a is the height of the clearance, and a_c the height of the contracted section. The pressure coefficients at any point of a clearance wall

$$\bar{p}=\frac{p-p_\infty}{\gamma q}$$

where p is the pressure acting in some point upon the clearance wall; p_∞ is the pressure upstream of the clearance and beyond its zone of influence; $q=v^2/2g$, v is the mean velocity in the clearance; γ is the specific weight of water; and g is the acceleration due to gravity.

The pressure coefficient in another form is:—

$$\bar{P}=\frac{p-p_\infty}{p_\infty-p_0}$$

here p_0 is the pressure at the clearance outlet (0 indicates degrees).

The clearance cavitation coefficient is:—

$$H_{cl}=\frac{p_\infty-p_v}{\gamma q}$$

where p_v is the water vapour pressure.

TABLE I

Type, model number	a mm.	l mm.	v m.sec^{-1}
P1–P3	2–2·05	20–60	7·93–14·20
P4–P6	3·1–3·0	40·3–80	7·80–14·0
P7–P10	10	50–300	6·0–10·0
P11–P13	8·45	50–100	5·9–11·0
P14–P16	10	100–250	6·0–10·0
P22–P24	5·3	25–100	5·7–11·4
S17–S20	10–10·05	50–300	8·0–14·0
S25–S27	5·3	25–100	7·6–13·4
DS 10°	1·4–2	26–40	10–16·4
DS 4°	2	40	10–14
DS 2°	1·4–2	26–40	15·3–17·3

The clearance cavitation coefficient is:—

$$H_{0cl}=\frac{p_0-p_v}{\gamma q}$$

H^* is the critical value of H, corresponding to the inception of cavitation.

Using the first letters of the substituted words or the characteristic notions as indices, then H_{cl} is the clearance cavitation coefficient when it is not necessary to distinguish the clearance types; H_p is the clearance type *P* cavitation coefficient; H_S is that of clearance type *S* and so on.

To characterise the extension of the cavitation area the distance l_{cav} from the beginning to the end of the cavitation area was taken and was considered as an integral stable part of the cavitation region. The relative lengths of the cavitation area were generally used, i.e. the lengths being correlated either to the clearance length l, $\lambda=l_{cav}/l$ or to the clearance height a, $\lambda=l_{cav}/a$.

The various cavitation stages were produced by regulating the pressure in the tunnel while keeping the velocity v constant. The cavitation inception was determined by the acoustical method and corresponded to its appearance.

Various stages of the blade-tip cavitation were denoted by the capital letters of the alphabet, since the

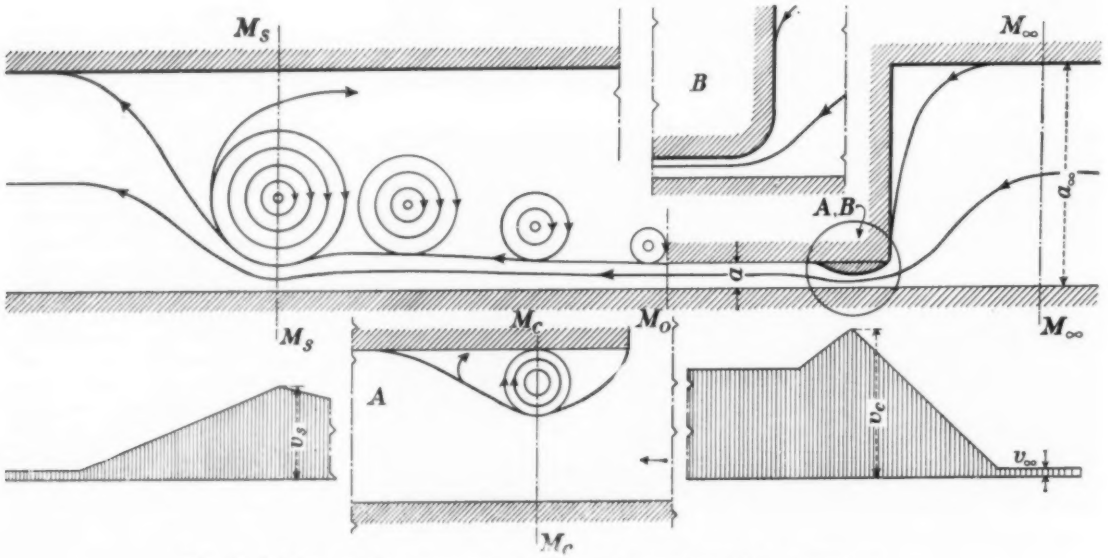


Fig. 5. Scheme of flow through clearance P, *used for* H* *calculations*

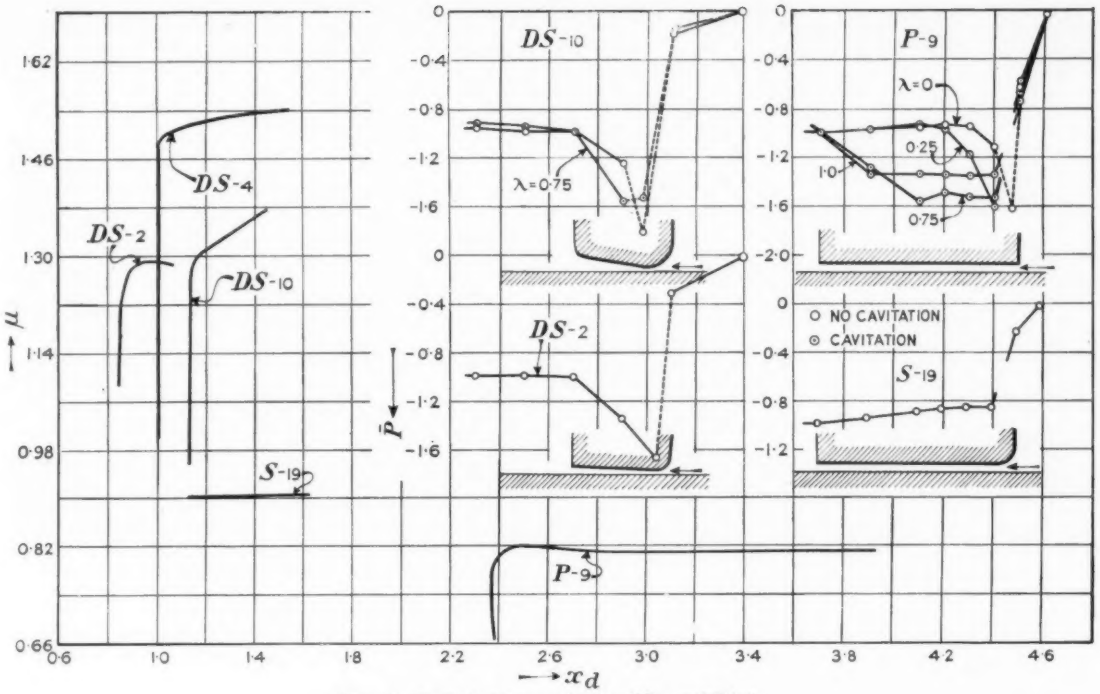


Fig. 6. Combined graph of μ(H) *and* P(x)

cavitation area length could not characterise any of the cavitation stages.

Let us consider the kinds of cavitation, in clearances of various types, that may be created with decreasing H_{cl} (Figs. 2, 3). Two kinds of cavitation, according to the writer's terminology, i.e. blade-face cavitation and blade-tip cavitation, were found to exist in clearances type *P*. As systematic observations had shown, the blade-face cavitation began near the clearance inlet at the contracting wall and took the form of a flashing cloud. Even before its actual appearance the cavitation was easily detected by sharp clicking noises. With decreasing H_{cl} the cavitation cloud became denser and extended in both directions, i.e. to the inlet and to the outlet of the clearance. When the cavitation zone end reached $\lambda=0{\cdot}75$ other kinds of cavitation started to form, i.e. blade-tip cavitation downstream the clearance. The blade-tip cavitation incepted at a significant distance from the outlet edge of the model, $l_{cav}=30-35a$, in the form of a rapidly-flashing cloud. With decreasing H_{cl} the cloud became denser and was observed at the limit where the mixing of the streams took place. The cloud grew towards the clearance outlet and at very low values of H_{cl} it almost reached the outlet edge.

The blade-face cavitation was not found in type *S* clearances. Only the blade-tip cavitation was observed and then only at lower values of H_{cl}*.

In clearances type *DS* various kinds of cavitation may occur depending on the angle, θ, of the contracting wall. In the clearance type *DS* at $\theta<3°$ two kinds of cavitation occur; namely, blade-tip cavitation behind the clearance model and surface-roughness cavitation on the clearance wall opposite the blade face. Experimenters are always surprised by the persistence of surface-roughness cavitation, as it seems to spring into existence as though it were a living thing. The presence of a hardly-noticeable scratch, or of a rust spot, is enough to cause cavitation long before the blade-tip cavitation takes place.

In the diffuser clearance *DS* with the angle $\theta>3°$, all three kinds of cavitation were found—blade-face cavitation, blade-tip cavitation and surface-roughness cavitation.

Values of H_{cl}* for clearances of the different types are shown in Table II. To understand the phenomenon better and make sure of the reliability of the test values of H_{cl}* shown in Table II, values of H_{cl}*′ are given, computed from the formulae which establish the relationship between H_{cl}* and the parameters of the flow through clearances without cavitation. These formulae, which will be described later, are also useful because they show how the model shapes and the flow parameters can affect H_{cl}*.

Comparing the photographs of the cavitation taken in the water tunnel, Fig. 3, with those taken in the water race, Fig. 4, one can see that the blade-face cavitation in clearance *P* and the blade-tip cavitation in clearance *S* occur in the vortex flow area. In agreement with the writer's previous investigations made with the aim of locating the inception of the round-profile burbling cavitation[18], let us suppose that here also the rupture of the liquid occurs first of all along the vortex axis.

Then using Bernoulli's equation, and applying it to the flow in clearances, it is possible to get formulae of the relationship with parameters, characterising a flow through a clearance without cavitation. In fact, for a flow from the section upstream of the clearance to the contracted section in the clearance (due to the compression of the stream flowing round the straight angle of the clearance inlet), it can be written (Fig. 5):

$$\frac{p_\infty}{\gamma}+\frac{v_\infty^2}{2g}=\frac{p_c}{\gamma}+\frac{1}{\alpha^2}\frac{v^2}{2g}+\frac{\zeta_1}{\alpha^2}\frac{v^2}{2g} \qquad ...(1)$$

where ζ_1 is an inlet resistance (drag) coefficient.

Taking into account that the pressure along the vortex axis is:—

$$\frac{p_0}{\gamma}=\frac{p_c}{\gamma}-\frac{v_c^2}{2g} \qquad ...(2)$$

Where $v_c=v/\alpha$ we get from (1) and (2), equating $p_{min}=p_0=p_v$, the following expression:—

$$\frac{p_\infty-p_v}{\gamma}=\frac{2}{\alpha^2}\frac{v^2}{2g}+\frac{\zeta_1}{\alpha^2}\frac{v^2}{2g}-\frac{v_\infty^2}{2g} \qquad ...(3)$$

Relating (3) to $q=1$, we have for the blade-face cavitation in clearance P:—

$$H_P^*=\frac{2+\zeta_1}{\alpha^2}-m^2 \qquad ...(4)$$

where $m=v_\infty/v$.

For the blade-tip cavitation in clearances S, using an analogous method, we deduce:—

$$H_S^*=2\frac{v_S^2}{v^2}-m^2+\zeta_2+\zeta_3 \qquad ...(5)$$

where ζ_2 is a clearance-resistance coefficient and ζ_3 is a flow-resistance coefficient downstream of the clearance and on its path from the clearance outlet up to the section where the first cavities occur.

In another form:—

$$H_{0S}^*=2\frac{v_S^2}{v^2}-1+\zeta_3 \qquad ...(6)$$

In deducing the formulae of H_{DS}^* for the type DS clearance, we must assume that in the model having the angle $\theta\leqslant 3°$, cavitation incepts "on the wall". Then

$$H_{DS}^*=1-m^2+\zeta_1 \qquad ...(7)$$

The value of H_P^* is related in a very simple way to the flow parameters α and ζ_1. As α and ζ_1 are nearly independent of the Reynolds number[19], the values of H_P^* for the clearances P should be constant within the range of velocities and the clearance sizes used in the writer's tests. The calculated values of H_P^* given in Table II were obtained at $\alpha=0{\cdot}7$ to $0{\cdot}75$ and $\zeta_1=0{\cdot}14$. The somewhat low values of H_P^* for clearances $P1$–$P6$ may be a result from deformation of the water tunnel chamber wall, and for this reason it is recommended that values of H_P^* be used for the model P7 and for models of other higher numbers.

The values of H_S^* for the S models depend mainly on the ratio v_S/v. The values of H_S^* for these models, calculated on the assumptions that $\zeta_2=\zeta_3=0$ and that $v_S/v=0{\cdot}8$ to $1{\cdot}0$ are included in Table II.

The significant difference between values of H_{DS}^* obtained by tests and by calculations, is found in these values for the diffuser clearances. This problem has not been studied in detail. It is possible that the cause is a boundary-layer thickness effect.

The effect of the cavitation in clearances of different types on the flow ratio μ and on the pressure $\bar{P}$ is illustrated by a combined graph for $\mu(H)$ and $\bar{P}(x)$, Fig. 6, where x is the distance between the pressure-measuring points and a reference section of the working chamber. All the typical clearance properties which must be taken into account in designing a blade are shown clearly in this graph. For example, the clearance $DS2$ gave very low values of H_{DS}^* for blade-tip cavitation; (the blade face cavitation was not found). However, this advantage is invalidated by the high value of the flow-rate ratio μ and of the pressure coefficient $\bar{P}$. The low pressure peak in the narrow section is the reason for the inception of the surface-roughness cavitation on the contracting wall.

TABLE II

Type, model, number	H_{cl}^*	$H_{cl}^{*\prime}$	Re $\times 10^{-4}$	μ
P P1–P3	3·30–2·70	3·82–4·36	1·48–2·64	0·824–0·812
P4–P6	3·27–3·42	,,	2·17–4·30	0·800–0·762
P7–P10	3·89–3·58	,,	6·13–10·31	0·838–0·794
P10–P13	4·00–3·71	,,	5·60–8·74	0·829–0·775
P14–P16	3·49–3·60	,,	6·46–11·10	0·827–0·808
P22–P24	3·89–3·52	,,	3·27–6·38	0·836–0·830
S17–S20	1·43–1·61	1·62–2·0	7·87–11·50	0·964–0·884
S25–S27	1·39–1·4	,,	3·87–6·74	0·975–0·942
DS 10°	1·38–1·45	1·03	0·68–1·17	1·54
DS 4°	1·25–1·55	,,	0·83–1·17	1·38
DS 2°	0·9–1·05	—	0·83–1·36	1·28

Note: H_{cl}^* is determined by tests, $H_{cl}^{*\prime}$ is calculated by formulas (4, 5, 7), at $\alpha=0{\cdot}75$ to $0{\cdot}70$.

(To be continued)

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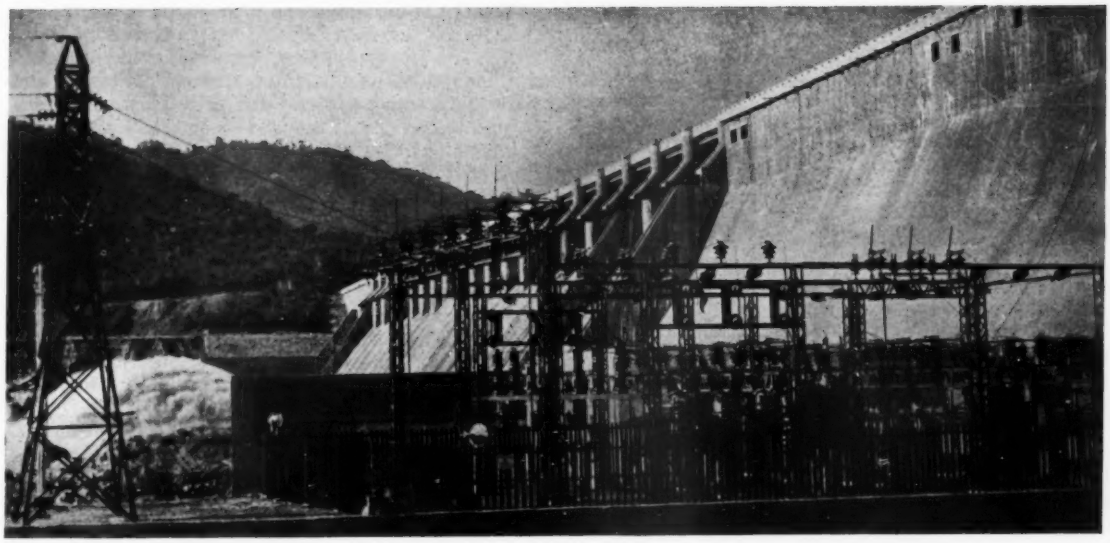


The Massenjore Station in West Bengal

A brief description of a multiple-purpose project on the River Mayurakshi is given by Dr. M. Datta

THE history of the Mayurakshi project dates back to 1928 when an investigation was initiated with the object of providing irrigation facilities in the Birbhum district, which has a sad record of periodic crop failures. The Mayurakshi is essentially a small stream which does not carry any appreciable volume of water except in the monsoon season. But its total annual flow, even in a year of drought, was considered sufficient to irrigate the major part of the area through which it flows if the monsoon discharges could be impounded. Extensive virgin forests and swamps constitute a natural holding ground for the rainfall and at Massenjore the river flows through a narrow valley where the conditions are very favourable for the construction of a dam. Here a stone masonry dam, 2,170 ft. long and 123 ft. high, has been constructed.

The catchment area consists of about 718 sq. miles of virgin forests and swamps, and the reservoir has a storage capacity of half a million acre-ft., sufficient to provide a steady discharge rate of about 300 cusecs for power-generation purposes, with an additional discharge of 300 cusecs during the monsoon period.

Power Station Equipment

The power station is situated immediately downstream of the dam, and consists of a stone masonry building measuring 55 ft. x 71 ft. The basement is at an elevation of 292 ft. and the roof at 331 ft. There are two turbines of the vertical Francis type which operate on a head ranging between 116 and 69 ft. With a head of 90 ft. and a utilisation rate of 300 cusecs each set develops 2,800 h.p. at 428 r.p.m., and generates 2,500 kVA at 11 kV. The alternators are so designed that they can withstand a runaway speed of 750 r.p.m. The stator has a diamond winding and two circuits per phase to permit split-phase relay protection, and the rotor is equipped with a low-resistance damping winding. The turbines were manufactured by Canadian Allis Chalmers and the alternators by Canadian English Electric.

Operation of Main Service and Substation

The power generated at 11·5 kV is stepped up to 34·5 kV in two 11·5/34·5 kV 2,500 kVA transformers, supplied by the Packand Electric Company of Canada, which are installed in an outdoor substation. Double-circuit overhead lines transmit the power to the more distant parts of the distribution area. The generators are controlled by two 150 MVA 11 kV Canadian Westinghouse circuit breakers having the usual protective devices. The alternator is earthed through a resistor of 54 ohms with a neutral isolating link. The step-up transformers are controlled by two 11 kV 150 MVA circuit breakers having overcurrent protection, differential protection with harmonic restraint, Buchholz gas and surge protection, intertripping with the respective 33 kV circuit breaker controlling the 33 kV transmission lines. To control the two outgoing 33 kV feeders two 33 kV 250 MVA standard rating, open-type single-break circuit breakers, each equipped with directional overcurrent and directional earth-fault protection, have been installed. All the oil circuit breakers in the outdoor substation are arranged for remote control from a desk-type control panel provided by Bedard Girand Limited of Canada. The scheme was financed under the Colombo Aid Plan, the aggregate cost being in the neighbourhood of Rs 30 lacs.

Book Reviews

Regime Behaviour of Canals and Rivers. By Prof. T. Blench, B.Sc. (Hons.), M.I.C.E., M.A.S.C.E., Professor, University of Alberta. Published by Butterworths Scientific Publications, 88 Kingsway, London, W.C.2. 138 pp., 11 dwgs. and 7 plates. Price 30s.

In striking contrast with the usual class of textbooks on hydraulics, dealing mainly with rigid channels, this interesting new book is devoted, solely and exclusively, to various problems concerning the flow of water in erodible channels, i.e., either man-made canals or natural rivers. It is chiefly based on Indian practice, and is the outcome of the author's extensive research work, before his retirement from the post of Director of Research, Punjab, India.

To explain the specific Indian meaning of the term "regime," which appears in the title of the book, we must start with Kennedy's basic discovery (1895) that shallower sections did not silt, or silted less than the then usual deeper canals. A logarithmic plot of his observations on the Upper Bari Doab canal system, yielded the curve $v_c = Cd^n = 0{\cdot}84\ d^{\ 0{\cdot}64}$ ft. per sec., in which v_c was the critical, non-silting (and non-scouring) velocity, whilst d was the depth. In combination with Kutter's formula, Kennedy's equation was employed for preparing design charts, which, for many years, were widely used on the Indian canal system.

At the same time, further research came to show that Kennedy's constants were not universally applicable to all Indian canal systems, and, therefore, a large number or other equations were suggested by different authors; e.g., Lindley's formulae $v_c = 0{\cdot}95\ d^{\ 0{\cdot}57}$ ft. per sec. and $v_c = 0{\cdot}57\ b^{\ 0{\cdot}355}$ ft. per sec. It seems that this author was the first one to have used the term "regime" in its specific sense; he said: "When an artificial channel is used to convey silty water, both bed and banks scour or fill, changing depth, gradient, and width, until a state of balance is attained at which the channel is said to be *in regime*."

By the end of the third decade of this century, the various "regime" equations were of so large a number that the United Provinces Government decided to put some order in the matter, and placed Mr. Gerald Lacey on special duty to collect and analyse all available data bearing on the problem, and submit proposals for practical design purposes.

His first job was to fit exponential equations, including the same simple exponent, viz. $n = \frac{1}{2}$, to all Indian and foreign data on silt stable-velocity. He thus obtained the equation $v_c = Cr^{\frac{1}{2}}$ in which r was the hydraulic mean depth. Comparing, then, the values of the parameter C (which he replaced by $\sqrt{f}$) with the fineness of the sediment carried by the respective channels, he found that the numerical value of this coefficient increased with the size of the particles. For instance, f was 1·44 for a coarse sand and 0·582 for the fine material carried by the Irrawaddy canals. This coefficient became subsequently well known as "Lacey's silt factor."

He also gave two other equations, $p = 2{\cdot}67\ \sqrt{Q}$ ft. and $S = f^{\ 5/3}/1{,}788\ Q^{\ 1/6}$, in which p was the perimeter, Q the discharge and S the slope.

It will be observed that Lacey's three equations yielded all the information required for the design of the canal, i.e. its hydraulic radius, perimeter and slope, depending on one arbitrarily chosen variable only: the discharge. They were approved for practical application, in India, by the Central Board of Irrigation, in 1934.

Attention will next be called to the fact that the "silt factor" appearing in these equations depended on the fineness of the silt load only, but not on its total weight. Sir Claude Inglis produced, later, another set of formulae, including this last factor also; and finally, the author of the book under review gave a method of his own, distinguishing between the materials forming the sides and the bed of the channel respectively. His equations incorporate, therefore, two empirical constants:—

The bed-sediment factor $F_b = v_c^2/d$, and
The side-sediment factor $F_s = v_c^3/b$.

For design purposes we have:—

$$b = \sqrt{F_b\ Q/F_s};\ d = \sqrt[3]{F_s\ Q/F_b^2}$$
$$\text{and } S = F_b^{\ 5/6}\ F_s^{\ 1/12}\ Q^{\ -1/6}(3{\cdot}63\ g/v^{\frac{1}{2}})$$

Two chapters in the book, dealing respectively with small and large bed-load canals, describe in full detail the practical application of these equations, whereas another chapter, entitled "Dynamical Aspects of Regime Forms," is an attempt at explaining them theoretically. Whilst some of such explanations are given as actual facts, in other cases the author is less definite, and presents his speculations in a less final form. For instance, in referring to his bed-sediment factor, he says: "The author suspects that, when an equation has been derived, it will show that v^2/d is a measure of uplift" (p. 61).

It will be observed that the foregoing review deals mainly with man-made canals, but the last three chapters of the book are concerned chiefly with natural rivers. Thus, chapter 5 contains a discussion on meandering, chapter 6 deals with various problems concerning breadth, depth and slope, whilst the last (chapter 8) is devoted to river models.

Broadly speaking, it may be regretted that not enough information is given in the book on Mr. Gerald Lacey's and Sir Claude Inglis's latest design formulae, for the reader might wish to know more about the arguments of these rival design methods before he adopts unconditionally the author's own solution.

The book is attractively presented and contains some excellent photograph reproductions. It is recommended to the attention of all those interested in canal design and river regulation.

S.L.

I.E.C. Specification No. 87: Glass Insulators for Overhead Lines with a Nominal Voltage of 1,000 V and Upwards. International Electrotechnical Commission, 1 rue de Varembé, Geneva, Switzerland. Price, Sw. Fr. 7·50.

This specification applies to glass insulators for overhead electric lines with a nominal voltage of 1,000 V and upwards. The arrangement of the contents follows that of the corresponding I.E.C. Specification No. 75 for porcelain insulators.

The Publication is divided into six chapters, the

first three of which deal with general questions, with definitions used in the specification, and with general rules for impulse tests, power-frequency tests and classification of tests. The classification of tests makes a distinction between three groups of tests: Group I, which is defined as those tests which are intended to verify those characteristics of the insulators which depend only on the shape and size of the insulator and of its accessories; Group II, which contains tests for the purpose of verifying the other characteristics of the insulator and the quality of the materials used; and Group III, which contains tests for the purpose of eliminating insulators with manufacturing defects. The tests in these three groups are described in Chapters IV to VI.

There are two Appendices, the first dealing with humidity correction factors for flashover voltages and the second with the method of producing artificial rain and of verifying its characteristics. A curve of correction factors for impulse and power-frequency flashover voltages is included.

La Mécanique des Roches (Rock Mechanics). By J. Talobre, Dunod, Paris, 1957, 444 pp., 161 ff., with a preface by Mr. D. Olivier-Martin, Directeur de l'Equipement de l'Electricité de France.

In recent years Rock Mechanics has developed as a new independent science, parallel to Soil Mechanics. Rock, a solid soil, is so different from the soft soils studied in Soil Mechanics that an entirely new approach is required. Rock Mechanics takes over from Soil Mechanics some methods of measuring strains, deformations and stresses, but this is about all. On the other hand Rock Mechanics has many intimate links with geology, but here also it must be stressed that the final aim of Rock Mechanics is a practical one; it examines how a rock can be classified and its properties studied in view of practical applications. Rock Mechanics is a typical engineering science.

Mons. J. Talobre, Chief Engineer of Electricité de France, well known for his many papers on the subject, has actively contributed to the development of the new science, and his treatise, the first to be published on Rock Mechanics, arrives at the very moment when such a publication is becoming indispensable. So much important research work has been done in different countries in the last 10 years, so many valuable papers published, that somebody had to concentrate on the heavy task of classifying all the material and publishing the main part of it as a handbook. Monsieur Talobre is the best man to do this, not only because of his personal contribution to the subject, but also because his position as chief engineer of Electricité de France (Project Department) gives him first-hand information on the latest methods used and results obtained by this important European producer of hydro power.

The book is divided in three main parts: (1) Rock Mechanics and Tests (Chapters II to VII); (2); Rock Mechanics and the Theory (Chapters VIII to XI); and (3) Rock Mechanics and its Applications (Chapters XII to XVII). In the first part the author deals with the physical properties of rocks (already dealt with in an introductory chapter I). Rock prestressing (chapter III), rock resistance and deformation (chapter IV), and rock resistance to the tools and explosives used for attacking rocks are studied in this Part One.

Part Two collates practically everything which has been said and written about the theory of rock resistance in the last 20 years, on the theory of foundation on rocks, and, most important chapter X, on the theory of concrete lining and steel lining. Obviously the modern theories on pressure distribution between lining and rock, the theory of roof bolting, etc., are dealt with at full length in this most important part of the book.

Excavation and blasting techniques are the subject of Chapter XII (Part three) when all the practical methods of attacking rock, blasting it, maintaining the tunnel open against rock pressure, reinforcing it provisionally or permanently, grouting the rock, etc., are described thoroughly.

This book will be the indispensable tool and companion of all those who have to design, build or supervise hydro-electric powerstations. Let us hope that this book will be translated soon into English.

CHARLES JAEGER.

Hirakud Project, a special issue of the Indian Journal of Power and River Valley Development, has been published as No. 10 of Vol. VI, 8½ in. by 11 in., 142 pp., 210 ff., 24 folding diagrams and maps, 25 tables. Price in India, printed on art paper, paper wrapper Rs. 6.

A comprehensive and detailed survey is given of this great project which, through its development of the Mahanadi River, will provide badly needed power and irrigation water to the Orissa State all the year round while, at the same time, bringing under control the devastating floods of the past. In his leading article "A Gift to the Nation," Mr. M. S. Thirumale Iyengar compares the future possibilities offered to Orissa to those which followed the commissioning of the great Dnieper Dam. The construction features of the Hirakud Dam itself, on which the whole scheme hinges, are reviewed at length (pp. 7-30) by A. R. Chellami, superintending engineer in charge of the main dam. This consists of earth, masonry and concrete sections aggregating 15,748 ft. in length, flanked by 32.275 ft. of earth dykes, which close five gaps in the hillocks of the left bank, and a single stretch of 35,500 ft. of similar dykes on the right bank. These dykes and the methods applied to their construction are described by Dr. R. P. Verma (pp. 31-38). Among many other articles of interest in this special issue, we should like to single out "Gates in the Harakud Dam Spillway" by N. K. Agrawal, in which all the problems encountered in the design, manufacture and erection of these large gates are reviewed at length (pp. 75-88), as well as the ten pages contributed by Dr. K. C. Thomas on the Power House (p. 89). The final chapter of the series, "Visitor's Guide to Hirakud" by R. Rangachari (pp. 136-142) includes a condensed description of the whole development. The diagrams and plans inserted in this publication are very good and should prove very helpful to readers who wish to make a detailed study of this interesting project.

Rapier 490. A well illustrated brochure (No. 2/432) has been issued by Ransomes & Rapier Limited to describe their 2½ cu. yard (2 cu. m.) Shovel Dragline, which is essentially a heavy-duty shovel or dragline fitted with Ward-Leonard control.

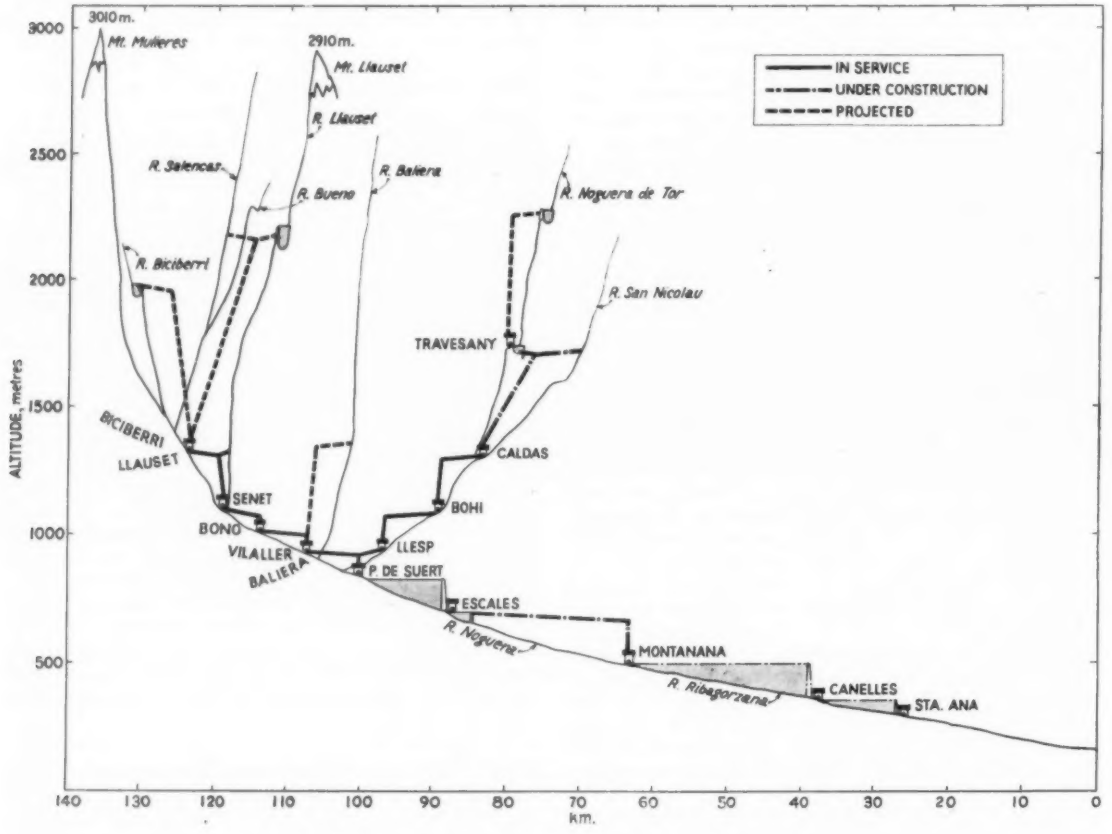
Developing the Ribagorzana Catchment

A brief outline is given of the ten years' work that has been done on this Spanish scheme, which aims at an integrated exploitation of the entire catchment area

IN 1946 the Spanish Government set up an organisation under the title of Empresa Nacional Hidroelectrica del Ribagorzana (ENHER), centred in Barcelona, to develop the Ribagorzana catchment in Catalonia, and a book recently published by ENHER to mark the completion of ten years of activity contains a most interesting record of work accomplished, in progress, and contemplated. The record is all the more creditable because, as we are given to understand, this is the first example in Spain of planning the development of an entire catchment, both for power and for irrigation, as an entity.

The Ribagorzana catchment lies to the south of the Pyrenees in the north-east of Spain, and for a considerable distance the river forms the boundary between the provinces of Huesca and Lérida. The headwaters rise in a circle of mountains reaching elevations of upwards of 3,000 m., and the flow leaves the power development for agricultural use at a level of 305 m. The catchment area is 2,036 sq. km. and is 130 km. long. The river rises under the foot of Mt. Mulieres (3,010 m.) and is joined in the vicinity by the Salencas and the Llauset. In the region of Ponte de Suert the Ribagorzana receives the waters from the Baliera and the Noruera de Tor, which in turn is fed by the San Nicolau. Ultimately the Ribagorzana becomes a tributary of the Ebro.

To harness these resources ENHER has planned 14 stations having an aggregate installed capacity of 478,400 kVA. After ten years seven of these stations are in service and four more are practically completed. Some 550 km. of 132 kV and 425 km. of 25 kV transmission line have been completed. The organisation has also established a cement factory, with a capacity of 200,000 tons per annum, in addition to roads and buildings such as dwelling houses, hostels, schools,



Profile diagram of the Ribagorzana development

and churches to serve the workers and their families.

A general idea of the scheme may be gathered from the profile diagram reproduced herewith, but the most important station of those already constructed is that at Escales. A concrete gravity dam, 125 m. high, stores 158 million cu. m. of water of which 120 million cu. m. are useful, the specific stored energy over the system being in excess of one kWh per cu. m. of water. This reservoir feeds an underground power house which has a capacity of 45 MVA under a maximum head of 117·5 m. The other six stations in service are of the surface type and have capacities ranging from 4·7 to 20 MVA.

Of the stations now under construction three are of the same order of capacity at Escales (Sta. Ana, 38 MVA; Caldas, 42·4 M.V.A.; Montañana, 56 MVA), but the fourth station, Canelles, will be the largest on the system. It will have an installed capacity of 133·8 MVA under a maximum head of 134 m. The dam, which will be 149 m. high by 210 m. crest length, will be one of the major double-curvature arch dams on the Continent, and will impound 576 million cu. m. of useful storage.

When the development is complete, the total available storage will be 936 million cu. m., corresponding to an energy equivalent of 566 million kWh. This is nearly as much as the average annual flow, which is 936 million cu. m. The average annual production will be 1,310 million kWh.

ENHER has also made proposals for the development of the Ebro between Escatrón and Flix which will contribute 1,900 million kWh in a normal year. This will be complementary to the Ribagorzana system as it will contribute its output mainly in the winter whereas the Ribagorzana catchment will function mainly in the spring and summer. A steam station at Escatrón will also be used to firm up the hydro power, and it is computed that the combined system will assure a yearly output of 3,500 million kWh under any hydrological conditions. This will be used primarily to serve Lérida, Tarragona and Barcelona, but connections will also be made to other systems.

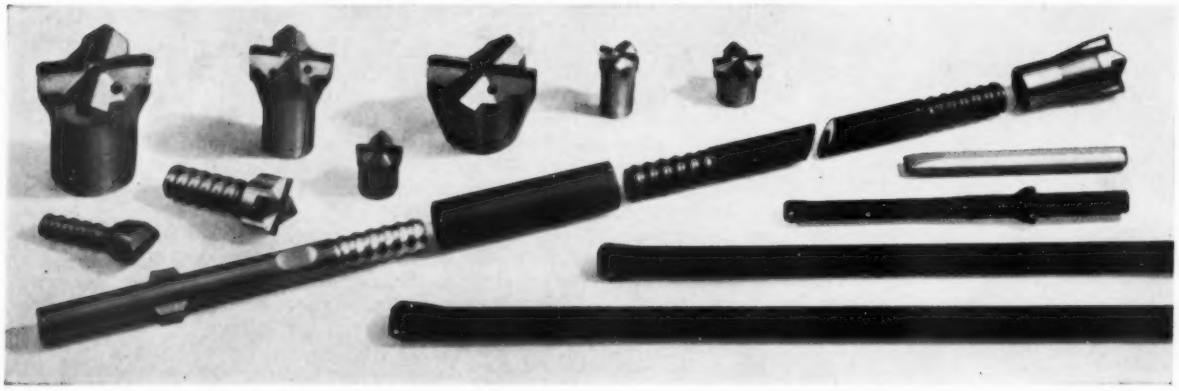
All-Purpose Loader

A new all-purpose loader known as the Loadall, has been developed by J. C. Bamford (Excavators) Limited, Lakeside Works, Rocester, Staffs., and has been introduced to supersede the J. C. B. Major Loader. In some respects this machine has retained the well-established principles of the Major Loader, but incorporates developments in hydraulic and structural design.

One of the main improvements is the incorporation of power crowd, giving complete control over the digging and discharging of materials and also enabling the operator to dig below ground level. Another feature is the use of the main structure as an oil tank. The design of the hydraulic system is such that the oil is drawn from one side of the main support right through the all-welded structure of the subframe to the rams, and back again to the reservoir through oil filters.

The main advantage of this system is that the greater circulation of oil ensures that it does not become overheated. A further advantage is the ease of detachment from the tractor, leaving the tractor available for other work.

The rams exert a breakout force of 4,000 lb. The Loadall has a maximum discharge height of 9 ft., a forward reach at maximum discharge height of 2 ft. 6 in., and a range of buckets from ½ cu. yard narrow to 1½ cu. yard full width.



SANDVIK COROMANT—

A Complete Range of Drill Steel Equipment

You know as well as we do the advantages of buying all your drilling equipment from one supplier. These advantages become still more evident if you buy from the Sandvik Range. The Sandvik Steel Works are the world's largest manufacturers of tungsten carbide for rock drilling. Their production covers integral steels, detachable bits, extension steels and stone working tools—all made of high-quality Swedish alloy steel, all fitted with the well-known Coromant tungsten carbide inserts.

Integral steels with 50% longer life

Sandvik Coromant integral steels have up to 50% longer rod life than ordinary steels, thanks to anti-corrosion SR-treatment, which protects them during transport, storage and actual drilling. In addition, air-tight plastic caps give bit and shank extra protection during transport and storage. They are available in these standard sizes:—

$\frac{3}{4}$″ hollow hexagon	1′4″-13′1″
$\frac{7}{8}$″ hollow hexagon	1′4″-21′0″
1″ hollow hexagon	2′6″-21′0″
Flexible drill steels	2′7″-31′6″

Precision-made rock bits

The threads of Sandvik Coromant (cross and X-design) bits are precision milled. The bits are so accurately manufactured that not only smoother drilling but *longer life* are ensured. Standard bit diameter sizes range from $1\frac{1}{2}$″ to $4\frac{1}{2}$″. The 773 bits (bottoming type) are available with GD400 and GD600 thread, or with $1\frac{1}{4}$″, $1\frac{1}{2}$″ and 2″ rope thread. The 776 bits, for standard shoulder-type drill rods, are available with threads ranging from $\frac{7}{8}$″ to $1\frac{11}{16}$″.

Efficient extension steels

The rope-threaded joints of Sandvik Coromant extension steels are solid and make joining and unscrewing extremely easy. Sizes available: $\frac{7}{8}$″ and 1″ hexagon steels, $1\frac{1}{4}$″ and 2″ round steels. A special feature of the $1\frac{1}{4}$″ equipment is the $\frac{1}{2}$″ flushing hole, about twice as large as most. This gives better cleaning of the bore hole and a higher rate of advance, reduces wear and risk of steels sticking. The 'cold rolling' technique makes this wider flushing hole possible *without any loss of strength.*

Wide variety of Stone Working Tools

A single plug hole steel made by Sandvik is capable of drilling up to 1000 holes, each about 3.9″. Sandvik Chisel Steels are made with rubber sleeves to reduce vibration and protect the worker. Sizes available: Plug Hole Drill Steels with bit diameters ranging from approx. $\frac{21}{32}$″ to $\frac{7}{8}$″. Chisel steels with bit diameters from approx. $\frac{5}{16}$″ to $\frac{25}{32}$″.

The World's foremost drilling unit

Sandvik Coromant extension and drill steels have been developed in close co-operation with Atlas Copco, manufacturers of rock drills and other compressed air equipment. The combination of Sandvik steels and Atlas Copco rock drills is the world's most widely used drilling unit—responsible for the drilling of more than one thousand million feet each year!

The Sandvik Coromant accessories featured here are sold and serviced throughout the world by the Atlas Copco Group of Companies. We have mentioned only the most important products and sizes: for further details, please contact your local Atlas Copco Company or Agent, or write to Atlas Copco AB, Stockholm 1, Sweden.

Atlas Copco *Manufacturers of Stationary and Portable Compressors, Rock-drilling equipment, Loaders, Pneumatic tools and Paint-spraying equipment.*

D5577

Waterproofing by Flexible Rendering

By S. CHAMPION, M.Sc., Ph.D., A.M.I.C.E., M.I.Struct.E.*

A 250 ft. aqueduct water-head of 75 ft. treated with Rubbtac preparations

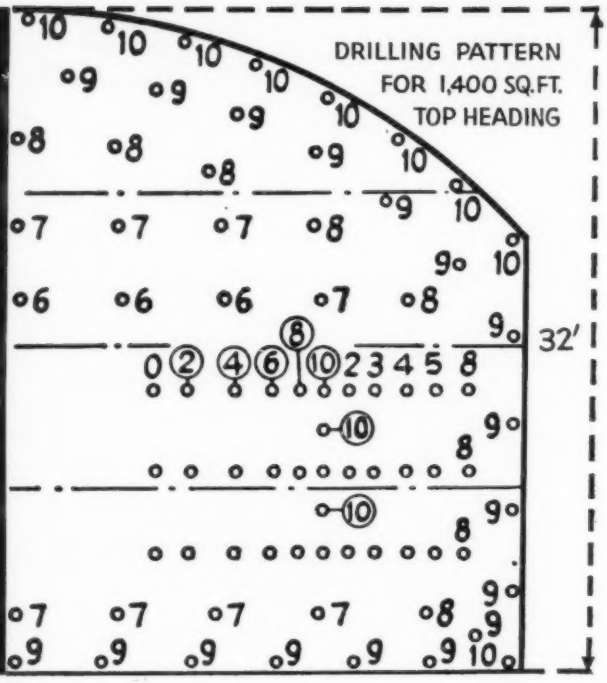
IN recent years the majority of waterproofing contracts carried out by Quickset Water Sealers Limited have been effected by applying a Setcrete sand-cement lining to the internal surfaces of the structure. Being rigid, the success of this type of lining depends on the rigidity of the structure itself, particularly as the pressure on it varies with the rise and fall of the water level. Although most engineering structures are designed to be completely stable against accumulated hydraulic pressure, there are instances where progressive deterioration has taken place and deflection occurs as the pressure rises and falls.

It is to meet such conditions that Quickset Water Sealers have developed a new flexible rendering that will not crack under deflective stresses. Asphaltic materials, though flexible, have sometimes shown certain deficiencies in these applications and a search was consequently made for a material that could be applied with the ease of asphalt, that was thermo setting (so that each day's work could be picked up on the following day), and that would be stable against a wide range of pH values. Other desiderata were resistance to any polluted ground water which might find its way from outside and attack the lining of, say, a sub-surface reservoir, freedom from curtaining on vertical surfaces, and immunity from attack by moulds, lichens and aquatic growths. A high hardening temperature was also desired, and adequate adhesive properties, especially in regard to old masonry, concrete, brickwork and metalwork. A further requirement was that this material should be able to span live cracks and joints without risk of failure and, of course, it would have to be reasonably priced in view of the large areas that need to be covered.

The search for a material meeting all these requirements ended some five years ago, when the final tests were completed on a compound to which the name Rubbtac has been given. This is essentially a rubber-based compound, the rubber being a reclaimed variety which is so treated until the final product is plastic rather than elastic. This means in practice that if the material is strained continually it will keep flowing continually but will recover when left alone. It can then be subjected to reverse strains without losing its original form—a very necessary requirement for a mastic used for spanning cracks in reservoirs and headwater canals where temperature conditions and pressure variations may cause movement.

Rubbtac is black in colour and begins to soften at about 150°C. At higher temperatures it never really melts into a liquid but becomes softened to a porridgy consistency, when it can be applied with the

* Managing Director, Quickset Water Sealers Limited.



Harrsele—a remarkable tunnelling operation in Sweden

Since the tunnelling operations began at Harrsele in northern Sweden the fast progress made there has been followed with great interest by construction and mining engineers throughout the world. On the 1,400 sq. ft. (32 ft. high, 49 ft. wide) top heading of the two-mile long discharge tunnel (the remaining bench to be excavated amounted to another 1,400 sq. ft.), the tunnelling crews beat the world tunnelling record in August 1955 with an advance of 585 feet, achieved by rounds of 26.4 feet. The work was carried on in three shifts; one for the drilling and blasting, and two for the mucking-out. On December 21, 1955, the break-through took place *—three months ahead of schedule.*

The 'Swedish Method' again!

The rock drills chosen for the Harrsele tunnel were exclusively Atlas Copco. Most of these were fitted with Sandvik Coromant steels. These lightweight drills and integral steels represent the basis of the 'Swedish Method'. To begin with, both were developed to work together so that drill and steel gave a completely balanced performance. Atlas Copco drills are small and compact. This means more drills at the face and a heavier concentration of drilling. The Sandvik Coromant steels with their new revolutionary SR anti-corrosion treatment have up to 50% longer life than untreated steels. The following drilling figures from the drifting at Harrsele show the impressive results to be gained from the 'Swedish Method'.

Advance per round	**26.4 feet**
Number of cu. yds. per round	**1,375**
Number of holes per round	**150**
Footage drilled per round	**3,940 feet**
Footage drilled per cu. yd.	**2.85 feet**
Number of drills and drillers	**21**
Number of man-hours per cu. yd. *(drilling, charging and blasting)*	**0.122**
Drilled footage per man-hour	**53.5 feet**
Advance per month *(27 working days)*	**580 feet**

Atlas Copco AR compressors supplied the air

The rock drills were fed from a battery of Atlas Copco AR 4E compressors. The famous AR-series give more air to the h.p. than other machines of their class.

World-wide sales and service

The Atlas Copco Group puts compressed air to work for the world. It is the largest group of companies specialising solely in the development and manufacture of compressed air equipment. It embraces Atlas Copco companies or agents manufacturing or selling and servicing Atlas Copco equipment in ninety countries throughout the world. For further details of the equipment featured here, contact your local Atlas Copco company or agent, or write to Atlas Copco AB, Stockholm 1, Sweden.

Atlas Copco *Manufacturers of Stationary and Portable Compressors, Rock-Drilling Equipment, Loaders, Pneumatic Tools and Paint-Spraying Equipment*

DSS 31

ordinary asphaltic techniques. Melting at such a high temperature, it also freezes at a similar temperature so that the work has to be carried out fairly quickly in order that it does not solidify before proper adhesion is obtained.

Thickness varying from $\frac{1}{8}$ in. to 4 and 5 in. have been successfully applied in different conditions. The thinner membranes are used on thoroughly stable surfaces where only waterproofing qualities are required, and the thicker applications have usually the object of sealing joints and cracks where a large differential movement is expected.

Types of Rubbtac

In a hydraulic retaining structure the escape of the contained liquid usually takes place by three quite distinct means: through simple cracks either large enough to permit a free flow or tight enough only to permit a seep; through tube-like flow paths formed through underwork or masonry joints or through segregated areas of concrete where the gravel, sand and cement have been badly mixed or compacted; and finally by a capillarity through the fine natural pores in the material.

To cater for these separate types of seepage three distinct forms of Rubbtac have been prepared. The first, for crack repair work, is the hot-poured thermosetting basic product Rubbtac. For the second a paste or putty-type variety is made, for application with a broad knife or steel float. For the third application a thinner solvent-type variety is made and applied as an impregnating surface dressing. Several coats of this solvent product act as a surface waterproof film, and also as a primer before the application of the hot-poured material.

These three types of Rubbtac are claimed to meet a very wide range of difficulties but the Company frankly admit that they have not been successful in getting first-class adhesion to a cold wet surface. Ventilation and local drying are therefore necessary. The primer form of Rubbtac, however, has good penetrating and good tack-coat properties. The solvent retention of rubber is much higher than that of asphalt so that the tack-coat and paste forms are relatively slow in hardening up. While this gives them superior properties in many respects it slows down the application on the job and the tack coats cannot be covered until the following day.

Site methods

In carrying out a contract a careful inspection survey is made in order to ascertain the location and manner of water seepage, giving particular attention to the water-level variations within the structure and also those of the external ground water in the case of an underground reservoir or aqueduct. Sometimes cracks up to 3 or 4 in. wide have been located, the contained water being retained by the presence of clay puddle.

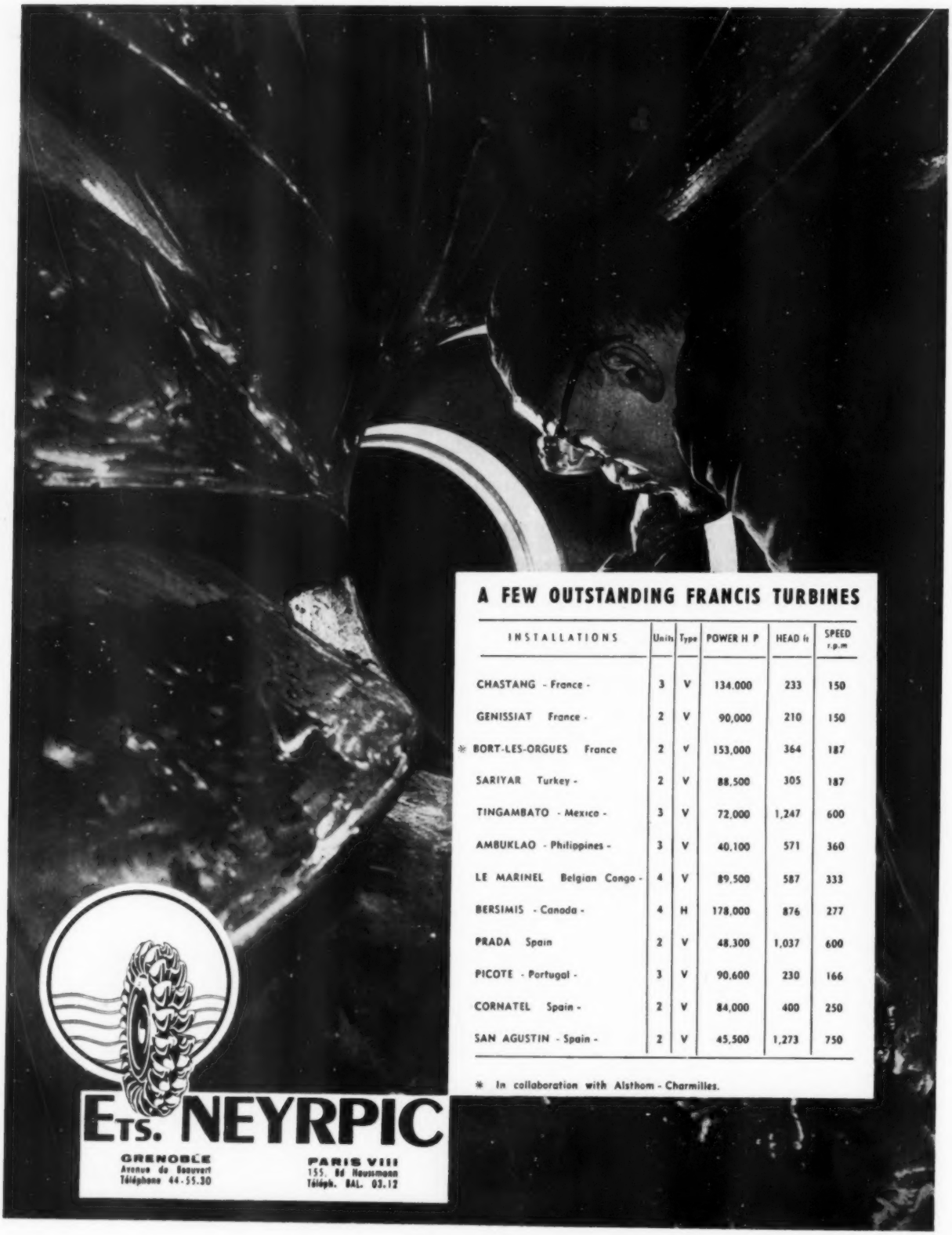
The surface to which the Rubbtac is to be applied is cleaned down, wire brushed and dried. The general dryness of the surface is first obtained by ventilation and as a secondary measure the immediate surface dryness adjacent to the men working is obtained by local blow lamping. The tack coat is applied cold by brush being well scrubbed into the surface and into every cavity of the surface. Subsequently pinholes, blowholes, bubbleholes and tiny cracks and other surface imperfections are buttered solid with the knifing coat or paste type. The blowlamp should not be used near these two solvent types of Rubbtac. Joints which are about $\frac{1}{16}$ in. wide can be spanned quite successfully with a $\frac{1}{4}$ in. blanket of the hot-poured material in which can be embedded a scrim of spun glass in one or two layers according to the differential movement expected. For larger cracks, of the order of $\frac{1}{2}$ in., and where the internal head may be some 15 ft. of water, an expanded metal blanket 4 in. wide can be placed across the crack to prevent the Rubbtac being forced into the crack under the pressure. For wider cracks of, say, 1 in. gap, and in circumstances where the internal water pressure may be 20 or 30 ft., it may be desirable to perform a piece of expanded metal into a horseshoe section and to thread the two edges into the crack and hammer it down level with the surface, thus forming an immovable support for the Rubbtac. In cases where expanded metal is used, spun-glass scrim cloth can be applied in the subsequent blanket of Rubbtac.

As a general rule it may be taken that the thickness of the blanket should be at least three times the width of the crack or at least three times the anticipated or possible movement of the crack, whichever is the greater. Finally, depending on the result of the inspection and the frequency and size of the cracking or jointing which has to be made good, the decision is taken either to render the surface completely with the hot-poured material or to prime and film the surface with the tack coat. In the case of a brick structure, where every joint is a potential source of weakness, it is usually most economic to apply a complete rendering of the hot-poured material, say, $\frac{1}{4}$ to $\frac{1}{2}$ in. thick. In old concrete structures where only the stop end joints are causing trouble it may be adequate to use $\frac{1}{4}$ in blankets over the potential sources of weakness and leave the general surface primed. Usually $\frac{3}{8}$ in. blankets are applied where concrete and metal meet and $\frac{3}{4}$ in. blankets where concrete and timber meet.

Quite a degree of structural knowledge is needed for the inspection and choice of method used for the repair but once the plan has been formulated the application does not require any particular degree of skill or experience.

Plant required

The tack coat is brushed directly from the container as delivered to the site, long-handled medium-stiff brooms, which can be cleaned in paraffin, being used for this purpose. The knifing coat is buttered out directly from the container with a broad knife, trowel or float. The hot-poured material, which is delivered in pieces the size of a tennis ball, can readily be heated in a cast-iron pot over a brazier or by means of a high-pressure blowlamp, and applied directly to the wall with a trowel and wooden float of the type used for asphalt work. Because of the high temperature of the molten Rubbtac care has to be taken to retain the heat and hence the use of cast-iron pots in place of buckets. The material chills rapidly when applied to a cold surface so the operative must work quickly to get a good degree of smoothness on the surface.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GENISSIAT France - | 2 | V | 90,000 | 210 | 150
* BORT-LES-ORGUES France | 2 | V | 153,000 | 364 | 187
SARIYAR Turkey - | 2 | V | 88,500 | 305 | 187
TINGAMBATO - Mexico - | 3 | V | 72,000 | 1,247 | 600
AMBUKLAO - Philippines - | 3 | V | 40,100 | 571 | 360
LE MARINEL Belgian Congo - | 4 | V | 89,500 | 587 | 333
BERSIMIS - Canada - | 4 | H | 178,000 | 876 | 277
PRADA Spain | 2 | V | 48,300 | 1,037 | 600
PICOTE - Portugal - | 3 | V | 90,600 | 230 | 166
CORNATEL Spain - | 2 | V | 84,000 | 400 | 250
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New B.I.C.C. Oil-filled Cable

FOR underground power transmission at high voltages, the oil-filled cable has been firmly established for many years, hundreds of miles having been installed on power system networks at home and overseas covering the complete transmission voltage range. In service, oil-filled cables have proved their reliability under arduous operating conditions and their usage is being continually extended.

The mass-impregnated or "solid" high voltage cable has voltage limitations and is prone to the formation of voids in the insulation which ionise and cause the dielectric to deteriorate. The oil-filled cable is completely impregnated with a thin mineral oil under all operating conditions and ionisation is consequently eliminated.

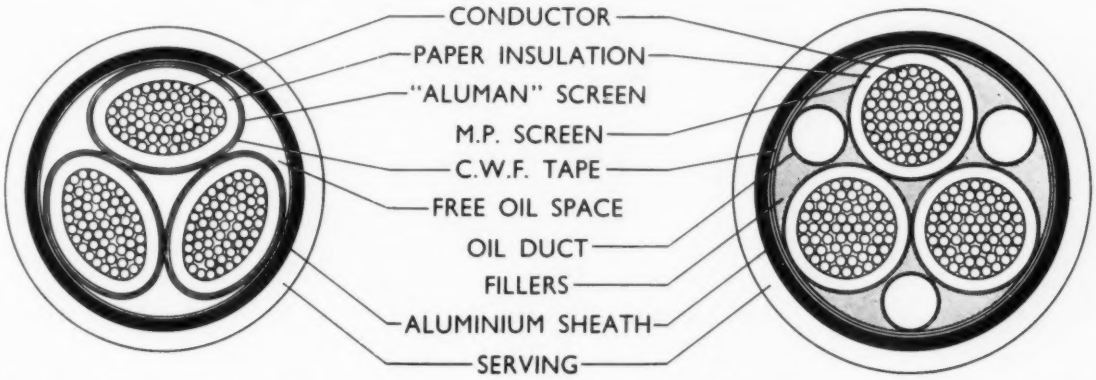
The most widely utilised system voltage for which pressure cables are employed is 33 kV, and although the conventional design of oil-filled cable at this voltage is economical, it was recognised that if certain techniques already established in other types of cable were extended to oil-filled designs, further savings in initial cost could be made.

Accordingly, British Insulated Callender's Cables Limited have developed a 33 kV ductless, shaped-conductor, oil-filled three-core cable (referred to as the D.S.O. cable) together with a range of joints and terminations which have passed the British Central Electricity Authority's test requirements.

In the D.S.O. cable the main innovation is the introduction of shaped conductors such as are commonly employed in 33 kV solid-type cables. It is the shaped cores which made a three-core ductless oil-filled cable possible, the free space between the cores being utilised for oil flow. Circular conductor oil-filled cables incorporate metallic spiral ducts for this purpose since they include paper fillers to give the sheath sufficient support to maintain its circular shape.

The cable is available with either copper or aluminium conductors over the range of sizes 0·20–0·60 sq. in. (125–390 sq. mm.) and either lead sheath or seamless aluminium sheath may be supplied. When a lead sheath is used metallic tapes are applied over the sheath to enable it to withstand the internal oil pressure. The aluminium sheath requires no such reinforcement owing to its greater inherent strength.

Anti-corrosion protection is achieved by an overall serving composed of a layer of self-bonding rubber tapes sandwiched between P.V.C. tapes and covered by a wrapping of bituminised hessian tape. This serving is available in several forms to meet particular cable or site requirements and is completely satisfactory under the most onerous conditions.



Section through B.I.C.C. ductless shaped-conductor oil-filled cable (left) compared with normal oil-filled cable (right)

The adoption of this design has resulted in cables of reduced diameter. This factor, together with the omission of built-in ducts and fillers and the lesser oil volume of the D.S.O. cable compared with the corresponding circular conductor cable, has naturally lowered the cost. The employment of an aluminium sheath, which requires no reinforcement, results, in general, in a still more economical oil-filled system.

The cable is designed to operate at a maximum conductor temperature of 85° C., this temperature having been the accepted standard for oil-filled cables in Great Britain for several years. The current ratings are similar to those of equivalent circular conductor oil-filled cables and are some 20 to 25 per cent. greater than those of equivalent 33 kV solid cables.

Contracts for the cable have been placed by several of the Electricity Boards in Great Britain, and in South Africa some 12 miles of D.S.O. cable having aluminium conductors and sheath are being installed for the City of Bloemfontein Electricity Department.

Models and Dioramas. Hunting Aerosurveys Limited have issued pamphlet HAS/57 to describe and illustrate models that have been constructed in their Model Department. Reference is made to the Rheidol hydro-electric project, interesting photographs being reproduced, side by side, of an aerial view of the actual river valley and of the model which has been constructed.

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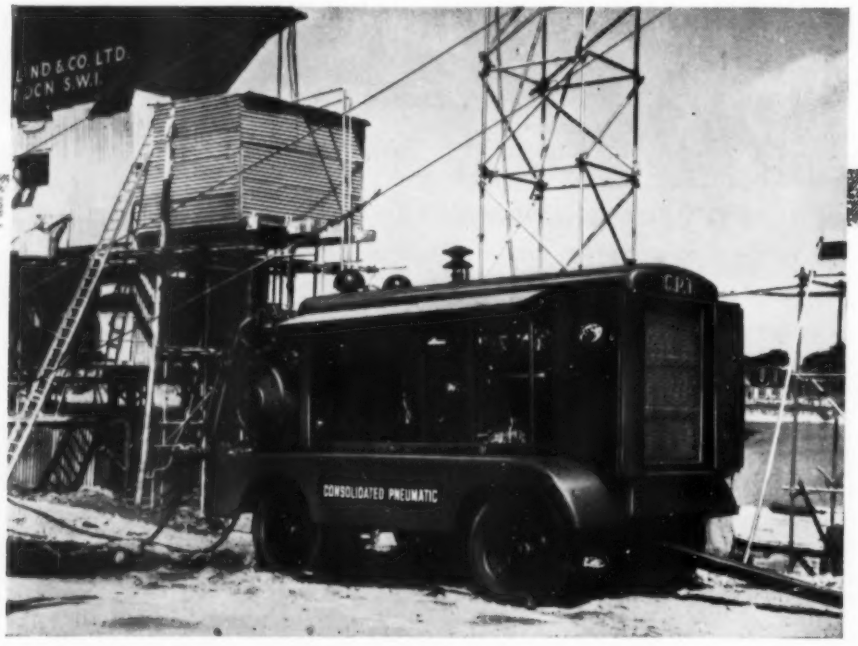
The outstanding advantages of operating "Power Vane" Oil Flooded Rotary Compressors have been proved by Messrs. Peter Lind & Co. Ltd., the well known London contractors. For over 3,800 working hours their Model 365-RO-2 "Power Vane" Rotary Compressor has been delivering air at 100 p.s.i., smoothly, effortlessly, without any major attention being required throughout the period of the Shoreham Harbour Contract. If you would like details of these "Power Vane" Rotary Compressors, which operate without pistons, valves, crankshaft or clutch, ask for Catalogue No. 58.



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Low-Deck Run-of-River Plants

A comprehensive critical survey is given based on a wide practical experience of this particular type of power plant whose main characteristic lies in the fact that the roof of the power house is placed at the same level as the top of the weir or spillway block, so as to form a flat deck from bank to bank. That this pattern of plant blends much more happily with the surrounding landscape than a high machine hall, clearly appears from a comparison of the photographic illustrations of two plants of about the same size: Birsfelden, near Bale, on the Upper Rhine, and Simback-Braunaun, one of the Inn River power stations. In the former, with its high power house, the view is fully blocked on the left bank of the Rhine, while it is in no way obstructed by the Simbach-Braunau structures. The advantages of this solution in this respect is even more marked at two other Inn plants, Stammhan and Neu-Oetting. The main objection put forward against the low-deck pattern is that it does away with the efficient shelter offered by conventional machine halls when initial erection work or subsequent overhaul or maintenance work on the generating sets is in progress. Electrical machinery is much more easily damaged by bad weather than a water turbine, and as the sets are fitted in individual pits, it is an easy matter to arrange for the erection of the generator to be postponed until after the erection of the turbine. Furthermore, by providing each generator pit with a roof which can either be rotated 180° on hinges or moved on rails, protection can quickly be arranged in case of sudden and unfavourable breaks in the weather. Experience with one-piece roofs has led the management of the Inn plants to adopt two-part sliding roofs, each half mounted on four rollers, the joint between the two halves being protected by a U-shaped rubber-lined strip. Another conspicuous advantage of the low-deck pattern is that the service cranes can be used as universal lifting gear, as they can run from end to end of the deck and thus attend to all requirements of the power and weir blocks. In plants equipped with more than two generating units, the author advocates the provision of two cranes, each having a lifting capacity equal to half the weight of the heaviest load occurring in practice. As a rule, the power block also includes, in addition to the generator pits, a roofed erection pit which can also be served by the cranes. An extension of the crane tracks beyond the deck, to connect with incoming or outgoing rail or road transport, is also highly desirable. Finally, as shown by the impressive view of the main service gallery of the 24 MW Stammhan plant, access to turbine governors, transformers and other ancillary equipment is fully ensured in a way which requires much less room than in high superstructures, while, on the other hand, large windows on the downstream side of the gallery afford in plenty of daylight. The experience accumulated in the eight Inn river plants, is so favourable to the low-deck type of power station that it has also been adopted for the completion of the Inn river development between Kufstein and Passau. As to the technical aspects, the low-deck plant is in no way inferior to the high-superstructure power house. On the economical side there is not, perhaps, much to choose between the two types in regard to building costs, but maintenance and overhaul work is definitely cheaper with the low-deck design. As yet, this type has been applied much more frequently in America than in Europe, but subject to the necessary topographical and other conditions being appropriate, there is no reason why it should not be more extensively adopted all over the world. (Dipl.-Ing. E. Treiber, Töging am Inn, *Schweizerische Bauzeitung*, Vol. 75, No. 14, April 6, 1957, p. 203, 6 pp., 11 ff.)

Note: A reprint of this article has been issued by Messrs. J. M. Voith G.m.b.H., Heidenheim (Brenz), Württemberg sub No. 1419.

Chainat Dam

This reinforced concrete structure, 305 m. long, is sited on the Menam Chao Phya ("Mother of Rivers") in the heart of Thailand, some 100 miles to the north of Bangkok and 6 km. downstream of the town of Chainat. It was officially declared open at the end of February 1957. Heading up 11 m. of water only, its primary purpose is irrigation and navigation. Following the decision to build another large multi-purpose dam at Yanhee, about 150 miles to the north of Chainat, which will provide a generating capacity of 560,000 kW, the original project of a 30,000-kW hydro-electric power house at Chainat has been postponed. (M. L. Xujati Kambhu, Director General, Royal Irrigation Department, Thailand, *Indian Journal of Power and River Valley Development*, Vol. VII, No. 5, May 1957, p. 13/14, 4 ff.)

King's River Development

This Pacific Gas and Electric Company's power development is a $80-million project located on the North Fork of the King's River in California and involves the use of three power plants. The uppermost work, the Courtright dam, located on Helms Creek, a tributary of the North Fork King's River, will create a reservoir of 102,000 acre-ft. capacity which will be used for carry-over storage. There is no power installation at this stage. Below, on the main stream is the Wishon dam, which will have an effective capacity at El. 6,550 of 128,000 acre ft.; here, the water level will be maintained at its maximum by releases from Courtright dam. In dry years, the whole of Courtright storage might be drained. The Wishon discharge will feed the new Haas power plant through a 6½-mile tunnel and a 4,650 ft. penstock. The Haas power house will have a 128,000 kW plant located in a solid-rock chamber 500 ft. below ground surface—the biggest underground power plant in the United States. The static head on the Haas turbines, with nozzles at El. 4,106, is 2,444 ft. Further downstream, the existing Balch power plant is fed from a diversion dam which will be increased from 103 to 148 ft. in height; the water will be carried through the existing 3½-mile tunnel to the power house, whose capacity will be raised to 126,000 kW through the addition of two new generating units. Below Balch, the existing 130 ft. high concrete arch diversion dam will be raised to 169 ft.

to divert the water, through a 3½-mile tunnel to the King's river power plant, which is to be located in the Pine Flat reservoir, which is a Corps of Engineers flood-control structure. While no contract has been yet placed for the latter plant, the Wishon tunnel and power house excavation is already in progress, and contracts have just been awarded for the Balch diversion dam enlargement and the Haas and Balch plants. (Larry Wise, ENR-Editor, *Engineering News-Record*, Vol. 159, No. 9, August 29, 1957, p. 44.)

Recent Developments in Fluid Mechanics

This paper attempts a survey of the knowledge available in four fields of fluid mechanics:— (1) sediment transport, (2) flow of water under dams built on a permeable foundation, (3) free boundary problems, (4) sedimentation by marine organisms. In the first three fields, the survey is supplemented by a description of recent work done by the author and his students at the National School of Engineering, Christchurch, New Zealand, and in the fourth, by a description of recent observations made in New Zealand. (F. M. Henderson, Lecturer in Civil Engineering, Canterbury University College, Christchurch, New Zealand, *New Zealand Engineering*, Vol. 12, No. 8, Aug. 15, 1957, p. 256, 16 pp., 26 ff.)

Intakes in Silt-Laden Streams

A short study is given of various patterns of the intake structures used in hydro-electric schemes which draw their water supply from silt-laden streams. Three types are dealt with: (a) lateral intakes, in which a branch canal diverts the water from the river to the generating plant; (b) weir intakes, in which the water is diverted into a canal from which it is separated by a pier, the canal being designed to act as a gravel-trap; (c) stream-bed intakes operating either independently (v. WATER POWER, July 1956, p. 282) or, to ensure undisturbed operation, in co-ordination with a branch canal, or with a diversion channel leading to a sand-trap. The fundamental advantages and drawbacks of each pattern are reviewed and discussed, the operating conditions assumed to be identical in all three instances. (Dozent Dipl.-Ing. Kâzim-Çeçen and Dipl.-Ing. Günther Garbrecht, Istanbul Technical University, *Die Wasserwirtschaft*, Vol. 47, No. 11, August 1957, p. 277, 6 pp., 12 ff.)

Piers in Run-of-River Plants

A short account is given of the tests carried out at the Bavarian State Laboratory for Hydraulic Structures, Obernach, with a view to establishing which shape of pier ensures the best flow conditions at any stage of operation of the turbines. Eight pier designs were tested. In two instances the length of the piers was increased, and in others pear-shaped and slotted piers were tried, and guide-walls were added. The novel type of pier designed by Ing. H. Hochstetter takes the form of the bow of a ship, that is, it overhangs at the top above the water. In comparative tests it is claimed that this overhanging pier gives the best results under widely operating conditions. When approaching the weir openings, the upper surface of the flow is hardly disturbed, the water does not separate from the pier wall, and there is no lowering of the water level along the inner sides of the pier heads. Altogether, the upper surface of the water is much quieter. (Ludwig Kirgis, *Die Wasserwirtschaft*, Vol. 48, No. 1, October 1957, pp. 15–16, 6 ff.)

Clutha Development

The undermentioned journal reproduces in full a paper read on January 9, 1957, before the Zurich Association of Engineers and Architects, which, in addition to general considerations of the climatic, topographical and geological conditions obtaining in New Zealand, includes a comprehensive description of the Roxburgh power station and dam. (Dipl.-Ing. Richar Pfenninger, AEBM, Mauvoisin, *Schweizerische Bauzeitung*, Vol. 75, No. 43, Oct. 26, 1957, p. 683, 9 pp., 24 ff.)

Water Power in Czechoslovakia

Under the title "Hydro-Electric Plants and their Latest Developments in Czechoslovakia," the English edition of the undermentioned journal gives an abundantly illustrated account of some of the latest hydro-electric developments in Czechoslovakia. It includes some of the works actually completed or in progress on the Vltava and on the Vah river, the latter being a left-bank tributary of the Danube. (Ing. Dr. F. Šembera, *Czechoslovak Heavy Industry*, Vol. 1956, No. 12, p. 3, 16 pp., 21 ff.)

Niagara Project

A few hours after the 2·2-million-kW Niagara power project had been approved by Congress, the Power Authority of the State of New York announced the first contract award, amounting to $20·7 million for the main turbines. Basic elements of the scheme are an intake structure on the U.S. side of the river above the Falls, 4 miles of covered conduit under the City of Niagara Falls, open unlined canal leading to the forebay of a 240,000-kW pumped storage plant, 1 mile of open canal which terminates at the forebay of the Lewiston power plant. The development will capture all but 1 ft. of the 315-ft. difference in water level between Lakes Erie and Ontario. This project, to be carried into effect at a cost estimated at $600 million, is also destined to be a speedily constructed one. All major equipment and construction contracts are to be awarded by the end of this year or shortly thereafter, and the power plant is to be on the line in the spring of 1960, just 30 months after groundbreaking. (*Engineering News-Record*, Vol. 159, No. 8, Aug. 22, 1957, pp. 27, 30 and 33, 2 ff.)

J. M. Voith, G.m.b.H., have sent us, together with an abridged version of the publication reviewed in our June issue of this year (p. 219), illustrated descriptions of the following hydro-electric plants for which they have supplied the turbines and ancillary equipment:—Cipreses, Chile, which utilises the water of Lake Invernada, in the Andes, 4,270 ft. above sea-level (net head 1,180 ft.), equipped with Pelton wheels; Marktbreit on the Main, Petershagen on the Weser, the Maxwehr plant of the Landshut Corporation (Bavaria), and three German run-of-river plants operating with Kaplan turbines of the well-known Voith design. At Petershagen, the turbines drive the horizontal-shaft generators and exciters by means of gearing.

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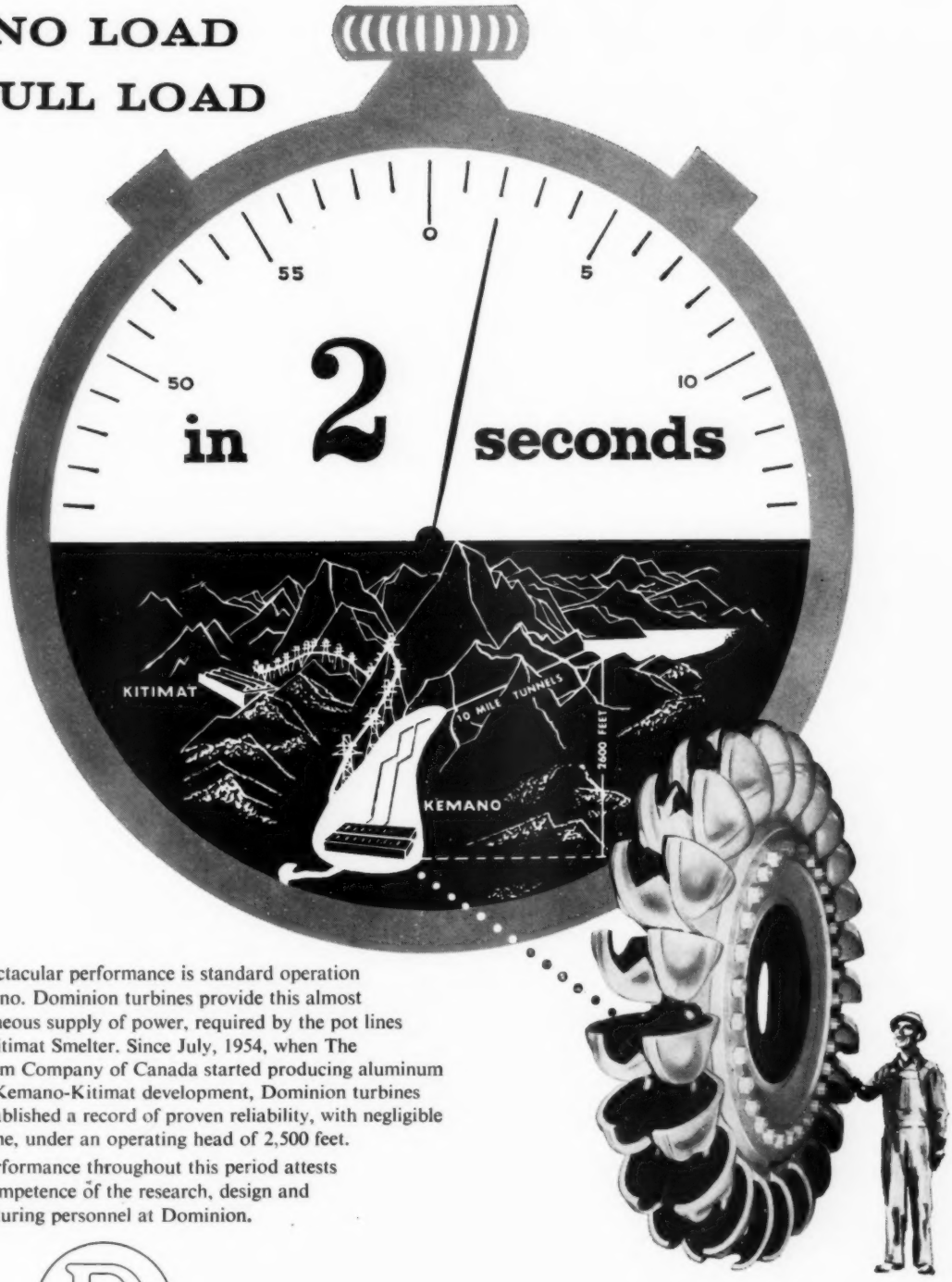

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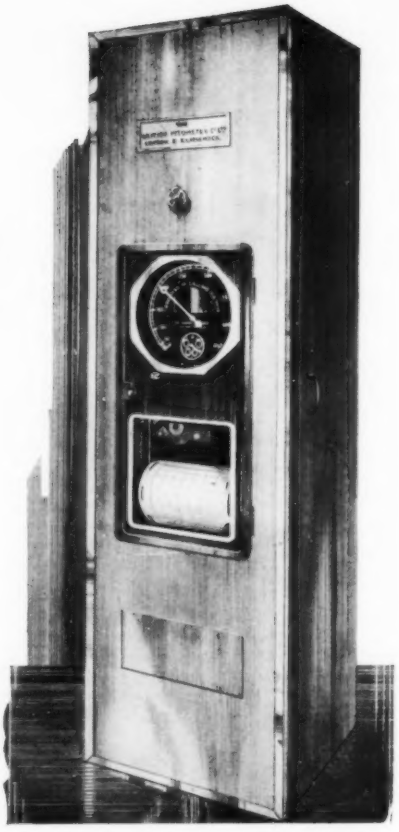

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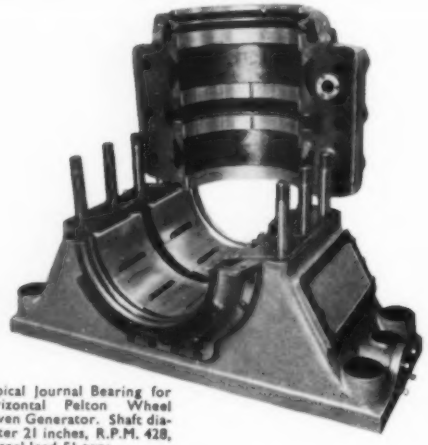

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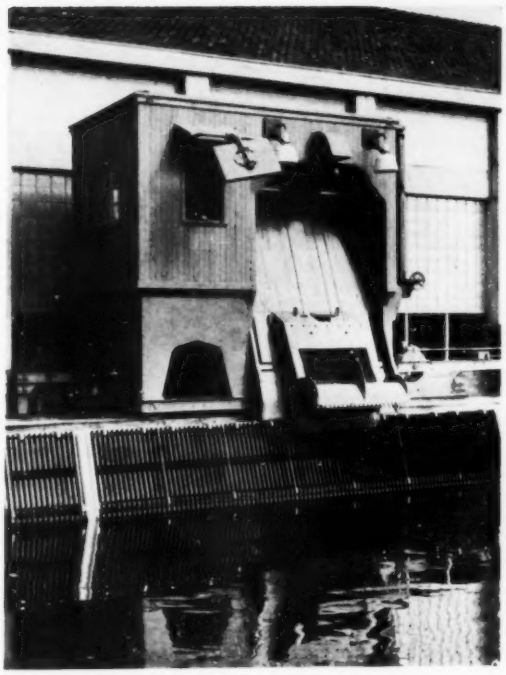

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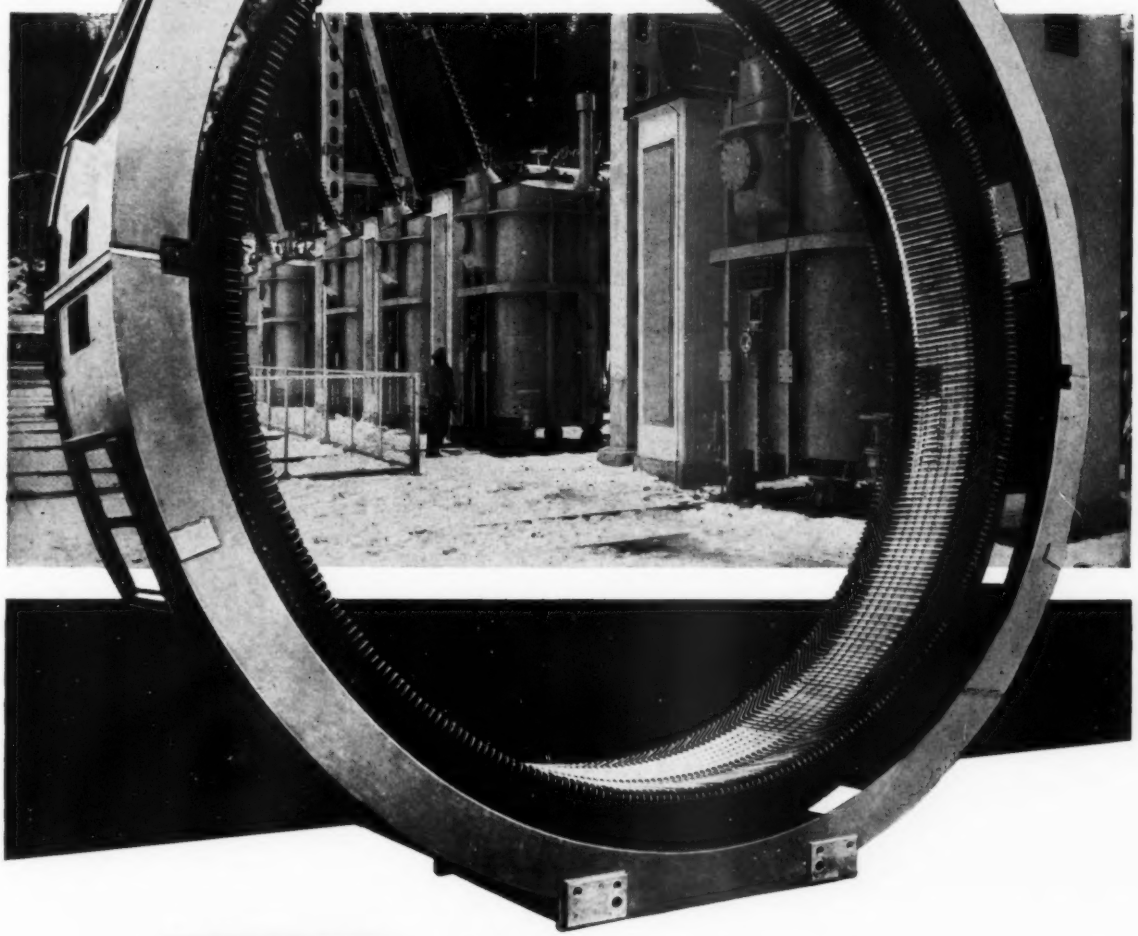
Hydro-Electric Power Station Equipment



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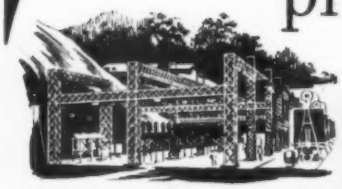
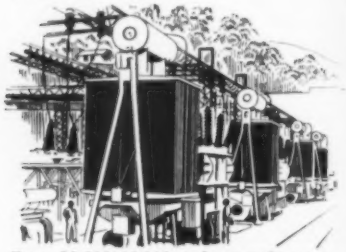
Comprehensive hydro-electric project at Sengulam

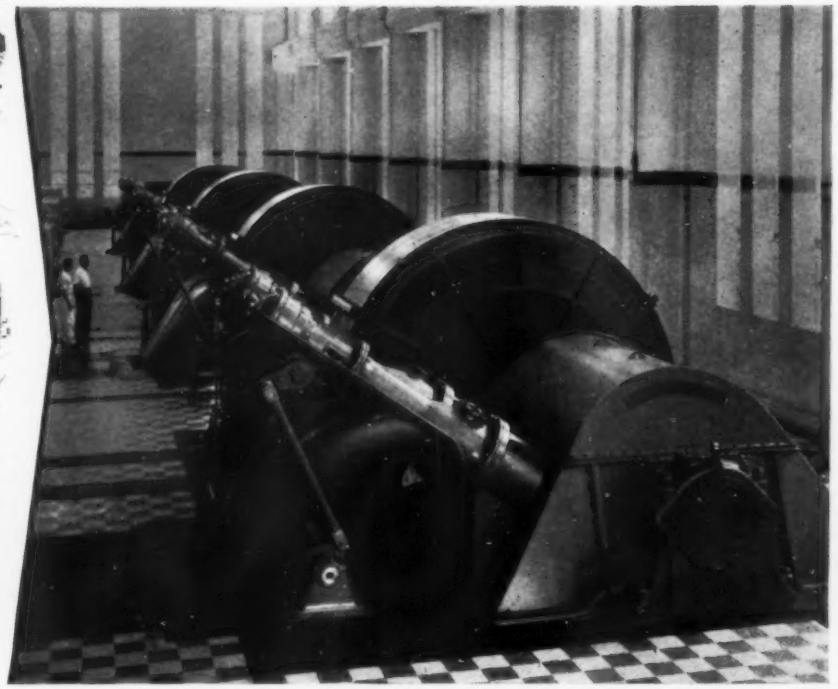


The outdoor 66,000 volt Switching Station at Sengulam.



Four 11,000/66,000 volt transformers at Sengulam.

The control desk and relay panel in Sengulam Power House.



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